STATE OF FLORIDA DEPARTMENT OF TRANSPORTATION

DRAINAGE HANDBOOK
OPEN CHANNEL

OFFICE OF DESIGN, DRAINAGE SECTION
TALLAHASSEE, FLORIDA

November 2009

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Chapter 1

Introduction

1.1 Background

The 1987 Florida Department of Transportation Drainage Manual was published as a three volume set: Volume 1 - Policy; Volumes 2A and 2B - Procedures; Volume 3 - Theory. On October 1, 1992, Volume 1 - Policy was revised to Volume 1 - Standards. With that revision, Volumes 2A, 2B, and 3 were designated as general reference documents. The Volume 1 - Standards were revised in January 1997 and were renamed to simply the “Drainage Manual”. No revisions have been, nor will be, made to Volumes 2A, 2B, and 3 of the 1987 Drainage Manual.

This handbook is one of several that the Central Office Drainage section is developing to replace Volumes 2A, 2B, and 3 of the 1987 Drainage Manual. In this form, the current Drainage Manual will be maintained as a “standards” document, while the handbooks will cover general guidance on FDOT drainage design practice, analysis and computational methods, design aids, and other reference material.

1.2 Purpose

This handbook is intended to be a reference for designers of FDOT projects and to provide guidelines for the hydraulic analysis and design of ditches and other open channel flows. Pertinent sections of the 1987 Drainage Manual have been incorporated into this handbook. The guidance and values provided in this handbook are suggested or preferred approaches and values, not requirements nor standards. The values provided in the Drainage Manual are the minimum standards. In cases of discrepancy, the Drainage Manual standards shall apply. As the Drainage Manual states about the standards contained in it, situations exist where the guidance provided in this handbook will not apply. The inappropriate use of and adherence to the guidelines contained herein does not exempt the engineer from the professional responsibility of developing an appropriate design.

1.3 Distribution

This handbook is available for downloading from the FDOT Drainage Internet site.

1.4 Revisions

Any comments or suggestions concerning this handbook may be made by e-mailing the State Hydraulics Engineer.

1.5 Terminology

An open channel is defined as any conduit conveying a fluid in which the liquid surface is subject to atmospheric pressure (i.e., has an open or free water surface). Open channel conditions are the basis for most hydraulic calculations. Pipe flow occurs in a conduit that is closed to atmospheric pressure and subject to hydraulic pressure alone. Pipe flow fundamentals, including open channel flow in pipes, are presented in the Storm Drain Handbook and the Cross Drain Handbook.

Flow problems in open channels are generally less easily solved than similar problems in pressure pipes. This is primarily because the physical conditions in channels, such as cross section, slope, and roughness, can vary considerably more. Calculations for open channel flow problems tend to be more empirical than those for pipes, and there is greater uncertainty when assigning friction factors for open channel flow.

Terminology needed to understand open channel flow problems is defined and discussed in the following sections. The key references for this information are Chow (1959) and Henderson (1966). Supplemental references include Streeter (1971), Simon (1981), and Rouse (1950), and HDS-3 (1961), HDS-4 (2001), and the Training and Design Manual (2001) from USDOT, FHWA.

1.5.1 Geometric Elements

Most open channel flow problems require an evaluation of various geometric elements associated with the shape of the channel. For most artificial or constructed open channels, geometric elements can be determined mathematically in terms of depth of flow and other dimensions for the channel shape. However, for most natural channel sections, profile sections based on the actual variations in the depth of flow across the section are generally required. The following geometric terminology is pertinent to the fundamentals of open channel hydraulics:

Prismatic channel. An artificial channel with non-varying cross section and constant bottom slope.

Channel section. The cross section of a channel taken perpendicular to the direction of flow.

Depth of flow. The vertical distance of the lowest point of a channel section from the free surface.

Stage. The elevation or vertical distance of the free surface above a given point.

Top width. The width of the channel section at the free surface. Refer to Figure 1-1.

Water area. The cross-sectional area of the flow perpendicular to the direction of flow. Refer to Figure 1-1.

<u>Wetted perimeter</u>. The length of the line of intersection of the channel wetted surface with a cross-sectional plane perpendicular to the direction of flow. Refer to Figure 1-1.

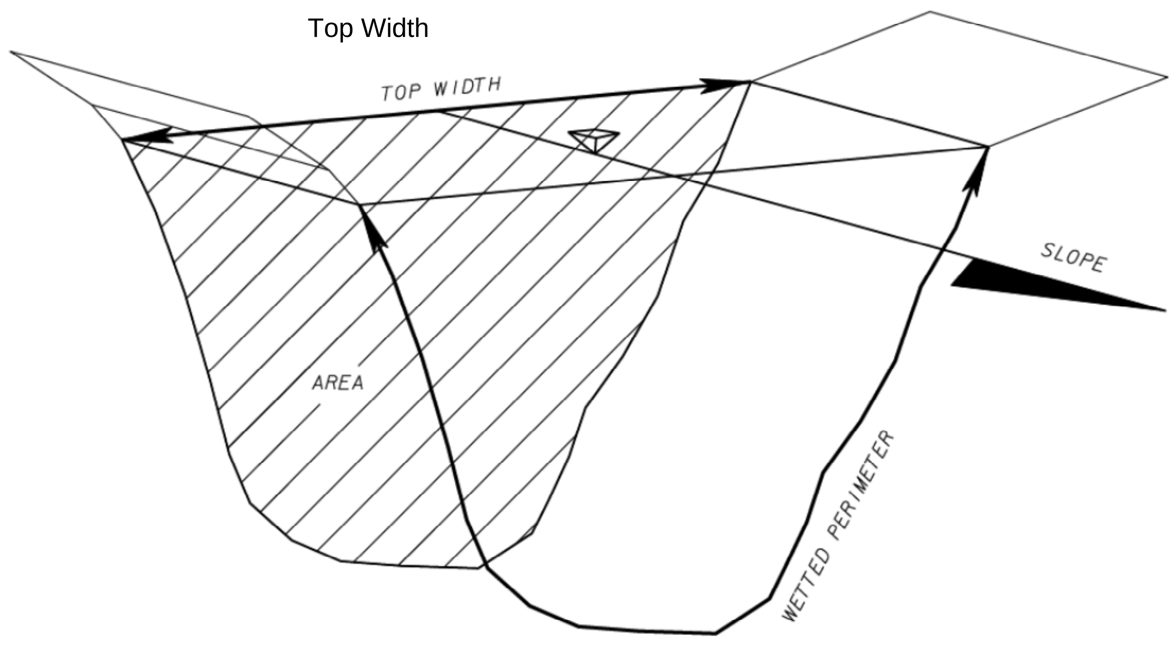


FIGURE 1-1
GEOMETRIC ELEMENTS

<u>Hydraulic radius</u>. The ratio of the water area to its wetted perimeter, which is expressed mathematically as:

$$R = \frac{A}{P} \tag{1-1}$$

where:

R = Hydraulic radius of the channel, in ft

A = Water area of the channel, in ft^2

P = Wetted perimeter of the channel, in ft

Hydraulic depth. The ratio of the water area to top width, which is expressed mathematically as:

$$D = \frac{A}{T} \tag{1-2}$$

where:

D = Hydraulic depth, in ft

A = Water area of the channel, in ft^2

T = Top width of the channel, in ft

Critical flow section factor. The ratio of the water area and the square root of the hydraulic depth, expressed mathematically as:

$$Z = \frac{A}{\sqrt{D}} = \frac{A}{\sqrt{A/T}} \tag{1-3}$$

where:

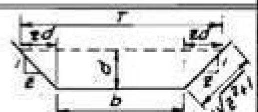
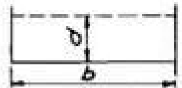
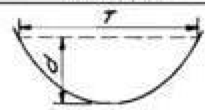
Z = Critical flow section factor

A = Water area, in ft^2

D = Hydraulic depth, in ft

T = Top width of channel, in ft

Figure 1-2 provides equations for the geometric elements of several commonly used prismatic channel shapes. Note that some of the variables in Figure 1-2 use different symbols than the equations given above. For example, water area is ‘a’ instead of ‘A’.

Section	Area a	Wetted Perimeter p	Hydraulic Radius r	Top Width T
 Trapezoid	$bd+zd^2$	$b+2d\sqrt{z^2+1}$	$\frac{bd+zd^2}{b+2d\sqrt{z^2+1}}$	$b+2zd$
 Rectangle	bd	$b+2d$	$\frac{bd}{b+2d}$	b
 Triangle	zd^2	$2d\sqrt{z^2+1}$	$\frac{zd}{2\sqrt{z^2+1}}$	$2zd$
 Parabola	$\frac{2}{3}dT$	$T+\frac{8d^2}{3T}$ [1]	$\frac{2dT^2}{3T^2+8d^2}$ [1]	$\frac{3a}{2d}$
 Circle - < ½ full [2]	$\frac{D^2}{8}\left(\frac{\pi\theta}{180}-\sin\theta\right)$	$\frac{\pi D\theta}{360}$	$\frac{45D}{\pi\theta}\left(\frac{\pi\theta}{180}-\sin\theta\right)$	$D\sin\frac{\theta}{2}$ or $2\sqrt{d(D-d)}$
 Circle - > ½ full [3]	$\frac{D^2}{8}\left(2\pi-\frac{\pi\theta}{180}+\sin\theta\right)$	$\frac{\pi D(360-\theta)}{360}$	$\frac{45D}{\pi(360-\theta)}\left(2\pi-\frac{\pi\theta}{180}+\sin\theta\right)$	$D\sin\frac{\theta}{2}$ or $2\sqrt{d(D-d)}$

[1] Satisfactory approximation for the interval $0 < \frac{d}{T} \leq 0.25$
When $d/T > 0.25$, use $p = \frac{1}{2}\sqrt{16d^2+T^2}+\frac{T^2}{8d}\sinh^{-1}\frac{4d}{T}$

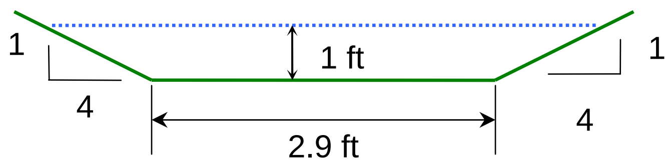
[2] $\theta = 4\sin^{-1}\sqrt{d/D}$
[3] $\theta = 4\cos^{-1}\sqrt{d/D}$
Insert θ in degrees in above equations

Reference: USDA, SCS, NEH-5 (1956).

FIGURE 1-2
Open Channel Geometric Relationships for Various Cross Sections

Example 1.1 – Geometric Elements

Given: Depth = 1.0 ft
Trapezoidal Cross Section shown below



Calculate: Area, Wetted Perimeter, Hydraulic Radius, Top Width, and Hydraulic Depth

Water Area

$$A = a = bd + zd^2$$
$$a = (2.9 \times 1) + 4(1)^2 = 6.9 \text{ ft}^2$$

Wetted Perimeter

$$P = b + 2d\sqrt{z^2 + 1}$$
$$P = 2.9 + (2 \times 1)\sqrt{4^2 + 1} = 11.146 = 11.1 \text{ ft}$$

Hydraulic Radius

$$R = r = \frac{bd + zd^2}{b + 2d\sqrt{z^2 + 1}}$$
$$r = \frac{(2.9 \times 1) + 4(1)^2}{2.9 + (2 \times 1)\sqrt{4^2 + 1}} = 0.619 = 0.62 \text{ ft}$$

Top Width

$$T = b + 2zd$$
$$T = 2.9 + (2 \times 4 \times 1) = 10.9 \text{ ft}$$

Hydraulic Depth

$$D = \frac{A}{T} = \frac{6.9}{10.9} = 0.63 \text{ ft}$$

This problem can also be solved using nomographs in Appendix A.

1.5.2 Steady / Unsteady Flow

Time variations of open channel flow can be classified as either steady or unsteady.

Steady flow occurs in an open channel when the discharge or rate of flow at any location along the channel remains constant with respect to time. The maintenance of steady flow in any channel reach requires that the rates of inflow and outflow be constant and equal. Conversely, open channel flow is unsteady when the discharge at any location in the channel changes with respect to time. During periods of stormwater runoff, the inflow hydrograph to an open channel is usually unsteady. However, in practice, open channel flow is generally assumed to be steady at the discharge rate for which the channel is being designed (i.e., peak discharge of the inflow hydrograph).

1.5.3 Uniform / Nonuniform Flow

Spatial variations of open channel flow can be classified as either uniform or nonuniform.

Uniform flow occurs only in a channel of constant cross section, slope, and roughness, known as a uniform open channel. If a given channel segment is uniform, the mean velocity and depth of flow will be constant with respect to distance. When the requirements for uniform flow are met, the depth of flow for a given discharge is defined as the normal depth of flow. In practice, minor variations in the channel bottom or deviations from the average cross section can be ignored as long as the average values are representative of actual channel conditions. Additional information about uniform flow is presented in Section 2.2.

True uniform flow rarely exists in either natural or artificial channels. Any change in the channel cross section, slope, or roughness with distance causes the depths and average velocities to change with distance. Flow that varies in depth and velocity when the discharge is constant, or steady, is defined as steady nonuniform flow.

Unsteady nonuniform flow, in which there are variations of both space and time, is the most complex type of flow to evaluate mathematically. Chow (1959) or Henderson (1966) should be consulted for details.

Nonuniform flow may be further classified as either rapidly varied or gradually varied. Rapidly varied flow is also known as a local phenomenon, examples of which include the hydraulic jump and hydraulic drop. The primary example of gradually varied flow occurs when sub-critical flow is restricted by a culvert or storage reservoir. The water surface profile caused by such a restriction is generally referred to as a backwater curve. Additional theoretical information on nonuniform flow evaluations is presented in Section 2.4.

1.5.4 Laminar / Turbulent Flow

Laminar flow generally occurs when the viscous forces are strong relative to inertial forces. Water particles will appear to move in definite smooth paths, or streamlines, when flow is laminar. Laminar flow is known to occur in shallow overland or sheet flow conditions.

When the viscous forces are weak relative to the inertial forces, the flow can be classified as turbulent. In turbulent flow, the water particles move in irregular paths which are neither smooth nor fixed, and the result is a random mixing motion. Since turbulent flow is the most common type occurring in open channel drainage facilities, it is the type considered for most hydraulic procedures.

1.5.5 Sub-Critical / Critical / Super-Critical Flow

The importance of gravity as a driving force in open channel drainage systems makes its effect on the state of flow a major factor for evaluation. This can be done using a dimensionless parameter known as the Froude Number, which is expressed mathematically as:

$$Fr = \frac{v}{(gL)^{1/2}} \tag{1-4}$$

where:

Fr = Froude Number, dimensionless

v = Average velocity of flow, in ft/sec

L = Characteristic length, in ft
(Hydraulic depth for open channels)

g = Acceleration due to gravity, 32.174 ft/sec^2

The Fr value is the dimensionless ratio of inertial forces to gravity forces. If Fr values are less than 1, gravity forces dominate and the open channel is said to be operating in the sub-critical range of flow. This is sometimes called tranquil flow and is characterized as relatively deep, low velocity flow with respect to critical flow conditions. Depth of flow is controlled at a downstream location.

If Fr values are greater than 1, inertial forces dominate and the open channel is said to be operating in the super-critical range of flow. This is also called rapid or shooting flow and is characterized as relatively shallow, high velocity flow with respect to critical flow condition. Depth of flow can be controlled at an upstream location.

When the Fr value equals 1, inertial forces and gravity forces are balanced and the open channel exhibits critical flow. Additional information on critical flow conditions is presented in Section 2.3.

Chapter 2

Open Channel Flow Theory

2.1 Mass, Energy, and Momentum

The three basic principles generally applied to flow analysis, including open channel flow evaluations are:

- Conservation of mass
- Conservation of energy
- Conservation of linear momentum

2.1.1 Mass

The conservation of mass for continuous steady flow is expressed mathematically in the Continuity Equation as:

$$Q = v \times A \tag{2-1}$$

where:

Q = Discharge, in ft^3/sec

A = Cross-sectional area, in ft ^{2}

v = Average channel velocity, in ft/sec

For continuous unsteady flow, the Continuity Equation must include time as a variable. Additional information on unsteady flow can be obtained from Chow (1959) or Henderson (1966).

2.1.2 Energy

The total energy head at a point in an open channel is the sum of the potential and kinetic energy of the flowing water. The potential energy is represented by the elevation of the water surface. The water surface elevation is the depth of flow, d, defined in Section 1.5.1 added to the elevation of channel bottom, z. The water surface elevation is a measure of the potential work that the flow can do as it transitions to a lower elevation. The kinetic energy is the energy of motion as measured by the velocity, v. If a straight tube is inserted straight down into the flow, the water level in the tube will rise to the water surface elevation in the channel. If a tube with a 90 degree elbow is inserted into the flow with the open end pointing into the flow, then the water level will rise to a level higher than the water surface elevation in the channel, and this distance is a measure of ability of the water velocity to do work. Newton's laws of motion can be used to determine that this distance is $v^2/2g$, where g is the acceleration due to gravity. Therefore the total energy head at a point in an open channel is $d + z + v^2/2g$.

As water flows down a channel, the flow loses energy because of friction and turbulence. The total energy head between two points in a channel reach can be set equal to one another if the losses between the sections are added to the downstream total energy head. This equality is commonly known as the Energy Equation, which is expressed as:

$$d_1 + \frac{{v_1}^2}{2g} + z_1 = d_2 + \frac{{v_2}^2}{2g} + z_2 + h_{loss} \qquad (2\text{-}2)$$

where:

d_1 and d_2 = Depth of open channel flow at channel sections 1 and 2, respectively, in ft

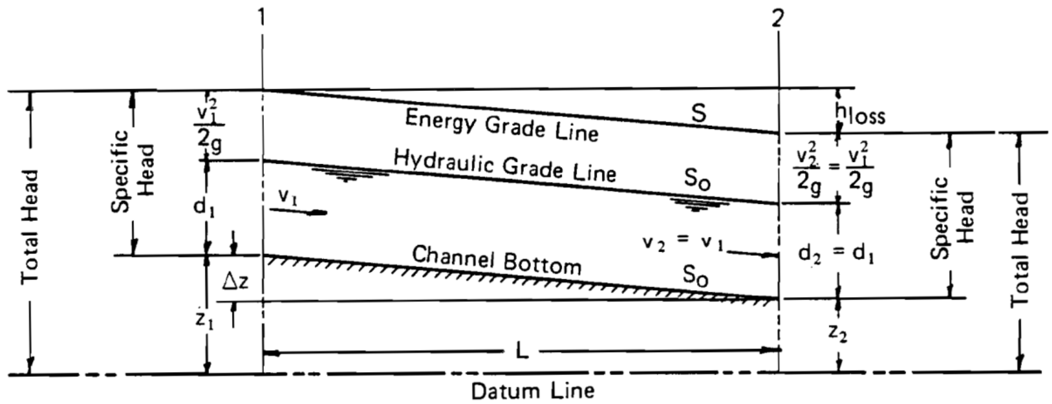
v_1 and v_2 = Average channel velocities at channel sections 1 and 2, respectively, in ft/sec

z_1 and z_2 = Channel elevations above an arbitrary datum at channel sections 1 and 2, respectively, in ft

h_{loss} = Head or energy loss between channel sections 1 and 2, in ft

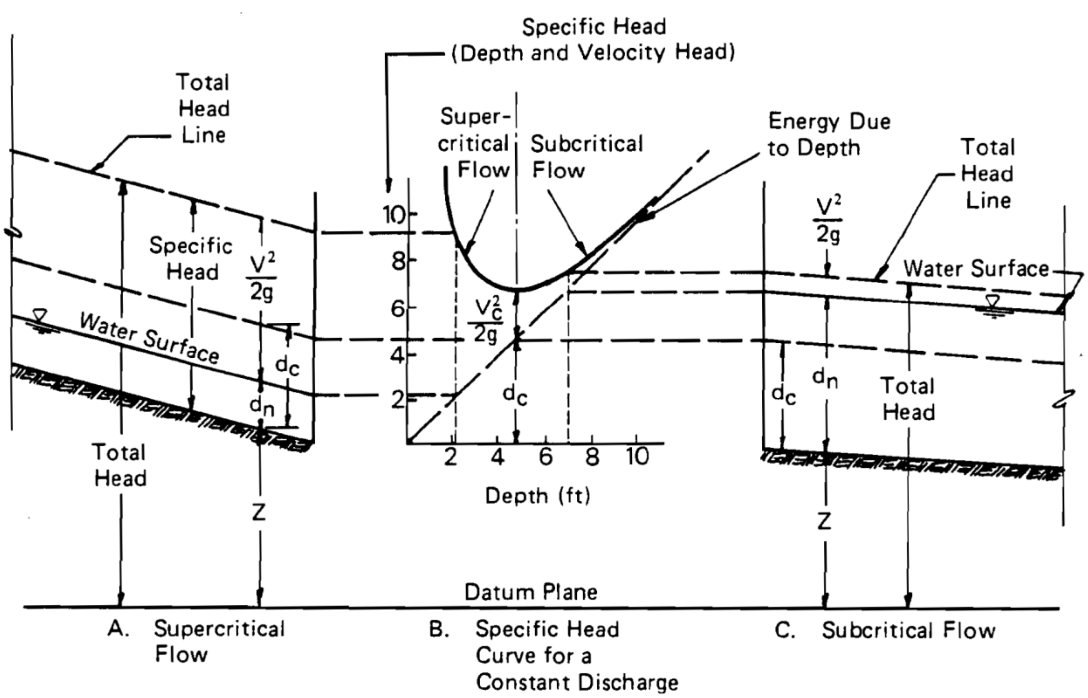
g = Acceleration due to gravity, 32.174 ft/sec^2

A longitudinal profile of total energy head elevations is called the energy grade line (gradient). The longitudinal profile of water surface elevations is called the hydraulic grade line (gradient). The energy and hydraulic grade lines for uniform open channel flow are illustrated in Figure 2-1. For flow to occur in an open channel, the energy grade line must have a negative slope in the direction of flow. A gradual decrease in the energy grade line for a given length of channel represents the loss of energy caused by friction. When considered together, the hydraulic and energy grade lines reflect not only the loss of energy by friction, but also the conversion between potential and kinetic forms of energy.



Reference: USDOT, FHWA, HDS-3 (1961).

FIGURE 2-1
Characteristics of Uniform Open Channel Flow



Reference: USDOT, FHWA, HDS-3 (1961).

FIGURE 2-2
Definition Sketch for Specific Head and Sub-Critical and Super-Critical Flow

For uniform flow conditions, the energy grade line is parallel to the hydraulic grade line, which is parallel to the channel bottom (see Figure 2-1). **Thus, for uniform flow, the slope of the channel bottom becomes an adequate basis for the determination of friction losses.** During uniform flow, no conversions occur between kinetic and potential forms of energy. If the flow is accelerating, the hydraulic grade line would be steeper than the energy grade line, while decelerating flow would produce an energy grade line steeper than the hydraulic grade line.

The Energy Equation presented in Equation 2-2 ignores the effect of a nonuniform velocity distribution on the computed velocity head. The actual distribution of velocities over a channel section are nonuniform (i.e., slow along the bottom and faster in the middle). The velocity head for actual flow conditions is generally greater than the value computed using the average channel velocity. Kinetic energy coefficients that can be used to account for nonuniform velocity conditions are discussed in the Bridge Hydraulics Handbook (not yet published).

For typical prismatic channels with a fairly straight alignment, the effect of disregarding the existence of a nonuniform velocity distribution is negligible, especially when compared to other uncertainties involved in such calculations. Therefore, Equation 2-2 is appropriate for most open channel problems. However, if velocity distributions are known or suspected to be non-typical, additional information related to velocity coefficients, as presented by Chow (1959) or Henderson (1966), should be obtained.

Equation 2-2 also assumes that the hydrostatic law of pressure distribution is applicable. This law states that the distribution of pressure over the channel cross section is the same as the distribution of hydrostatic pressure; that is, that the distribution is linear with depth. The assumption of a hydrostatic pressure distribution for flowing water is valid only if the flow is not accelerating or decelerating in the plane of the cross section. Thus, Equation 2-2 should be restricted to conditions of uniform or gradually varied nonuniform flow. If the flow is known to be rapidly varying, additional information, as presented by Chow (1959) or Henderson (1966), should be obtained.

2.1.3 Momentum

According to Newton's Second Law of Motion, the change of momentum per unit of time is equal to the resultant of all external forces applied to the moving body. Application of this principle to open channel flow produces a relationship which is virtually the same as the Energy Equation expressed in Equation 2-2. Theoretically, these principles of energy and momentum are unique, primarily because energy is a scalar quantity (magnitude only) while momentum is a vector quantity (magnitude and direction). In addition, the head loss determined by the Energy Equation measures the internal energy dissipated in a particular channel reach, while the Momentum Equation measures the losses due to external forces exerted on the water by the walls of the channel. However, for uniform flow, since the losses due to external forces and internal energy dissipation are equal, the Momentum and Energy Equations give the same results.

Application of the momentum principle has certain advantages for problems involving high changes of internal energy, such as a hydraulic jump. Thus, the momentum principle should be used for evaluating rapidly varied nonuniform flow conditions. Theoretical details of the momentum principle applied to open channel flow are presented by Chow (1959) and Henderson (1966). Section 2.4.3 provides a brief presentation of hydraulic jump fundamentals.

2.2 Uniform Flow

Although steady uniform flow is rare in drainage facilities, it is practical in many cases to assume that steady uniform flow occurs in appropriate segments of an open channel system. The results obtained from calculations based on this assumption will be approximate and general, but can still provide satisfactory solutions for many practical problems.

2.2.1 Manning's Equation

The hydraulic capacity of an open channel is usually determined through application of Manning's Equation, which determines the average velocity when given the depth of flow in a uniform channel cross section. Given the velocity, the capacity (Q) is calculated as the product of velocity and cross-sectional area (see Equation 2-1).

Manning's Equation is an empirical equation in which the values of constants and exponents have been derived from experimental data of turbulent flow conditions. According to Manning's Equation, the mean velocity of flow is a function of the channel roughness, the hydraulic radius, and the slope of the energy gradient. As noted previously, for uniform flow, the slope of the energy gradient is assumed to be equal to the channel bottom slope. Manning's Equation is expressed mathematically as:

$$v = \frac{1.486}{n} R^{2/3} S^{1/2} \qquad (2\text{-}3)$$

or

$$Q = \frac{1.486}{n} A R^{2/3} S^{1/2} \qquad (2\text{-}4)$$

where:

v = Average channel velocity, in ft/sec

Q = Discharge, in ft^3/sec

n = Manning's roughness coefficient

R = Hydraulic radius of the channel, in ft, calculated using Equation 1-1

S = Slope of the energy gradient, in ft/ft

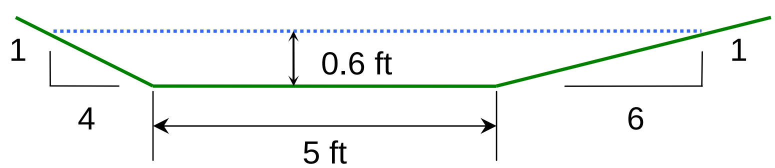
A = Cross-sectional area of the open channel, in ft^2

Design values for Manning's roughness coefficient for artificial channels (i.e., roadside, median, interceptor, and outfall ditches) are given in Chapter 2 of the Drainage Manual. Methods to estimate Manning's roughness coefficient for natural channels are included in the Bridge Hydraulics Handbook (not yet published).

Example 2.1 – Discharge given Normal Depth

Given: Depth = 0.6 ft
Longitudinal Slope = 0.005 ft/ft
Trapezoidal Cross Section shown below
Manning's Roughness = 0.06

Calculate: Discharge, assuming normal depth



*Not to scale

Note: Example 1-1 was solved using equations from Figure 1-2. This example will be solved using geometry. To make things easier, try breaking the drawing into 3 parts: 2 triangles and a rectangle.

<u>Step 1: Calculate Wetted Perimeter and Cross-sectional Area</u>

Wetted Perimeter (P):

Solve for the left triangle's hypotenuse

$$x = \sqrt{0.6^2 + (4 \times 0.6)^2}$$

$$x = 2.474 \text{ ft}$$

Solve for the right triangle's hypotenuse

$$x = \sqrt{0.6^2 + (6 \times 0.6)^2}$$

$$x = 3.650 \text{ ft}$$

Wetted Perimeter (P) $= 2.474 + 3.650 + 5 = 11.124$ ft

Cross-sectional Area (A):

Solve for left triangle's area

$$A_1 = \frac{1}{2}(4 \times 0.6)(0.6)$$

$$A_1 = 0.72 \text{ ft}^2$$

Solve for right triangle's area

$$A_2 = \frac{1}{2}(6 \times 0.6)(0.6)$$

$$A_2 = 1.08 \text{ ft}^2$$

Solve for rectangle's area

$$A_3 = 5 \times 0.6$$

$$A_3 = 3 \text{ ft}^2$$

Cross-sectional Area (A) = $0.72 + 1.08 + 3 = 4.8 \text{ ft}^2$

<u>Step 2: Calculate Hydraulic Radius</u>

Hydraulic Radius (R) = $\frac{A}{P}$

Hydraulic Radius (R) = $\frac{4.8}{11.124} = 0.4315 \text{ ft}$

<u>Step 3: Calculate Average Velocity</u>

Average Velocity (v) = $\frac{1.486}{n}(R)^{2/3}(S)^{1/2}$

Average Velocity (v) = $\frac{1.486}{0.06}(0.4315)^{2/3}(0.005)^{1/2} = 1.00$ ft/sec

<u>Step 4: Calculate the Discharge</u>

Discharge (Q) = $v \times A$

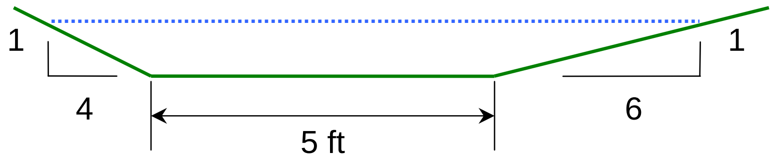
Discharge (Q) = 1.00 ft/sec $\times$ $4.8 \text{ ft}^2 = 4.80 \text{ ft}^3$/sec

Example 2.1 has a direct solution because the depth is known. The next problem will be more difficult to solve because the discharge will be given and the normal depth must be calculated. The equations cannot be solved directly for depth, so an iterative process must be used to solve for normal depth. Example 2.1 can also be solved using the charts in Appendix A.

Example 2.2 – Normal Depth given Discharge

Given: Discharge = 9 ft^3/sec
Use the Channel Cross Section Shape, Slope, and Manning's Roughness given in Example 2.1.

Calculate: Normal Depth



Note: The solution must use trial and error since the equations cannot be solved implicitly for depth. The first trial will be performed in the steps below and the remaining trials will be shown in a table. The initial trial depth (i.e., the first guess) should be greater than the depth given previously in Example 2.1 because the discharge is greater. So we will perform our trial with an estimated depth of flow of 0.8 ft.

<u>Step 1: Calculate Wetted Perimeter and Cross-sectional Area</u>

Wetted Perimeter (P):

Solve for the left triangle's hypotenuse

$$x = \sqrt{0.8^2 + (4 \times 0.8)^2}$$

$$x = 3.298 \text{ ft}$$

Solve for the right triangle's hypotenuse

$$x = \sqrt{0.8^2 + (6 \times 0.8)^2}$$

$$x = 4.866 \text{ ft}$$

Wetted Perimeter (P) $= 3.298 + 4.866 + 5 = 13.164$ ft

Cross-sectional Area (A):

Solve for left triangle's area

$$A_1 = \frac{1}{2}(4 \times 0.8)(0.8)$$

$$A_1 = 1.28 \text{ ft}^2$$

Solve for right triangle's area

$$A_2 = \frac{1}{2}(6 \times 0.8)(0.8)$$

$$A_2 = 1.92 \text{ ft}^2$$

Solve for rectangle's area

$$A_3 = 5 \times 0.8$$

$$A_3 = 4 \text{ ft}^2$$

Cross-sectional Area (A) $= 1.28 + 1.92 + 4 = 7.2 \text{ ft}^2$

Step 2: Calculate Hydraulic Radius

Hydraulic Radius (R) = $\dfrac{A}{P}$

Hydraulic Radius (R) = $\dfrac{7.2}{13.164} = .547$ ft

Step 3: Calculate Average Velocity

Average Velocity (v) = $\dfrac{1.486}{n}(R)^{2/3}(S)^{1/2}$

Average Velocity (v) = $\dfrac{1.486}{0.06}(0.547)^{2/3}(0.005)^{1/2} = 1.171$ ft/sec

Step 4: Calculate the Discharge

Discharge (Q) = $v \times A$

Discharge (Q) = 1.171 ft/sec $\times$ 7.20 ft$^2 = 8.43$ ft^3/sec

The discharge calculated in Step 4 is still less than 9 ft^3/sec, so normal depth is greater than 0.8 feet. The next depth of flow guess should be slightly higher. The following table summarizes subsequent trials. The trial and error process continues until the desired accuracy is achieved.

Depth (ft)	Area	Perimeter	Radius	Velocity	Discharge
0.8	7.2	13.16469	0.546917	1.171	8.433
0.85	7.8625	13.67499	0.574955	1.211	9.521
0.82	7.462	13.36881	0.558165	1.187	8.859
0.826	7.54138	13.43005	0.56153	1.192	8.989

The normal depth for the given channel and flow rate is 0.83 feet. Intermediate calculations should be performed with more significant digits than needed, and then rounded in the last step to avoid rounding errors.

The FDOT Drainage Manual recommends that if the flow depth is greater than 0.7 feet, the roughness value should be reduced to 0.042. However, the normal depth using n = 0.042 is 0.69 feet. The recommended roughness for flow depths less than 0.7 feet is 0.06. This anomaly is caused by the abrupt change in the recommended roughness values. If the flow depth is the primary concern, then using n = 0.06 will give a conservative answer. However, if the velocity is the primary concern, then using n = 0.042 is conservative.

2.3 Critical Flow

The energy content of flowing water with respect to the channel bottom is often referred to as the specific energy head, which is expressed by the equation:

$$E = d + \frac{v^2}{2g} \qquad (2\text{-}5)$$

where:

E = Specific energy head, in ft

d = Depth of open channel flow, in ft

v = Average channel velocity, in ft/sec

g = Acceleration due to gravity, 32.174 ft/sec^2

Considering the relative values of potential energy (depth) and kinetic energy (velocity head) in an open channel can greatly aid the hydraulic analysis of open channel flow problems. These analyses are usually performed using a curve showing the relationship between the specific energy head and the depth of flow for a given discharge in a given channel that can be placed on various slopes. The curve representing specific energy head for an open channel is generally used to identify regions of super-critical and sub-critical flow conditions. This information is usually necessary to properly perform hydraulic capacity calculations and evaluate the suitability of channel linings and flow transition sections.

2.3.1 Specific Energy and Critical Depth

A typical curve representing the specific energy head of an open channel is illustrated in Figure 2-2 (Part B). The straight diagonal line on this figure represents points where the depth of flow and specific energy head are equal. At such points the kinetic energy is zero; therefore, this diagonal line is a plot of the potential energy, or energy due to depth. The ordinate interval between the diagonal line of potential energy and the specific energy curve for the desired discharge is the velocity head, or kinetic energy, for the depth in question. The lowest point on the specific energy curve represents flow with the minimum content of energy. The depth of flow at this point is known as the critical depth. The general equation for determining the critical depth is expressed as:

$$\frac{Q^2}{g} = \frac{A^3}{T} \qquad (2\text{-}6)$$

where:

Q = Discharge, in ft^3/sec

g = Acceleration due to gravity, 32.174 ft/sec^2

T = Top width of water surface, in ft

A = Cross-sectional area, in ft^2

Critical depth for a given channel can be calculated through trial and error by using Equation 2-6. Chow (1959) presents a procedure for the analysis of critical flow which uses the critical flow section factor (Z) as defined by Equation 1-3.

Using the definition of the critical section factor and a velocity distribution coefficient of one, the equation for critical flow conditions is:

$$Z = \frac{Q}{\sqrt{g}} \tag{2-7}$$

where:

Z = Critical flow section factor
(see Equation 1-3)

Q = Discharge, in ft^3/sec

g = Acceleration due to gravity, 32.174 ft/sec^2

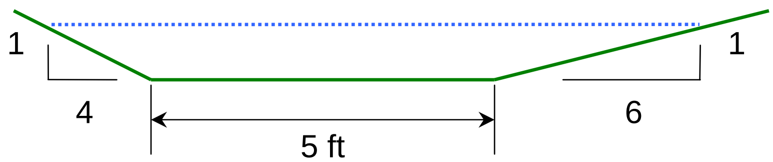
When the discharge is known, Equation 2-7 gives the critical section factor and thus, by substitution into Equation 1-3, the critical depth. Conversely, when the critical section factor is known, the discharge can be calculated with Equation 2-7.

It is important to note that the determination of critical depth is independent of the channel slope and roughness, since critical depth simply represents a depth for which the specific energy head is at a minimum. According to Equation 2-6, the magnitude of critical depth depends only on the discharge and the shape of the channel. Thus, for any given size and shape of channel, there is only one critical depth for the given discharge, which is independent of the channel slope or roughness. However, if Z is not a single-valued function of depth, it is possible to have more than one critical depth. For a given value of specific energy, the critical depth results in the greatest discharge, or conversely, for a given discharge, the specific energy is a minimum for the critical depth.

Example 2.3 – Critical Depth given Discharge

Given: Discharge = 9 ft^3/sec
Cross Section and Roughness from Example 2.1

Calculate: Critical Depth



Note: The solution must use trial and error since the equations cannot be solved implicitly for depth. The first trial will be performed in the steps below and the remaining trials will be shown in a table. Typically the slope of a typical roadside ditch channel must exceed 2% to have a normal depth that is super critical. Since the slope Example 2.1 and 2.2 is 0.5%, the critical depth is probably much less than the normal depth of 0.83 feet calculated in Example 2.2 for 9 cfs. So we will perform our trial with an estimated depth of flow of 0.4 ft.

Step 1: Calculate Cross-sectional Area

Cross-sectional Area (A):

Solve for left triangle's area

$$A_1 = \frac{1}{2}(4 \times 0.4)(0.4)$$

$$A_1 = 0.32 \text{ ft}^2$$

Solve for right triangle's area

$$A_2 = \frac{1}{2}(6 \times 0.4)(0.4)$$

$$A_2 = 0.48 \text{ ft}^2$$

Solve for rectangle's area

$$A_3 = 5 \times 0.4$$

$$A_3 = 2 \text{ ft}^2$$

Cross-sectional Area (A) = $0.32 + 0.48 + 2 = 2.8 \text{ ft}^2$

Step 2: Calculate Top Width

Top Width (T):

Base Length of Left Triangle + Bottom Width + Base Length of Right Triangle

$$(4 \times 0.4) + 5 + (6 \times 0.4) = 9 \text{ ft}$$

Step 3: Rearrange Equation 2-6 to solve for Discharge, then solve

$$\frac{Q^2}{g} = \frac{A^3}{T}$$

$$Q^2 = \frac{A^3}{T} \times g$$

$$Q = \sqrt{\frac{A^3}{T} \times g}$$

$$Q = \sqrt{\frac{2.8^3}{9} \times 32.174} = 8.86\,\text{ft}^3\text{/sec}$$

The discharge calculated in Step 3 is less than 9 ft^3/sec, so critical depth is greater than 0.4 feet. The next depth of flow guess should be slightly higher. The following table summarizes subsequent trials. The trial and error process continues until the desired accuracy is achieved.

Depth (ft)	Area (sq. ft.)	Top Width	Discharge (cfs)
0.4	2.8	9	8.858665864
0.45	3.2625	9.5	10.84467413
0.41	2.8905	9.1	9.2404111
0.404	2.83608	9.04	9.010440628

This problem can also be solved by determining the minimum specific energy as discussed in the previous section. The following table solves Equation 2-5 for depths bracketing the critical depth determined above, and shows that the critical depth has the minimum specific energy.

Depth (ft)	Area (sq. ft.)	Perimeter (ft)	Velocity (ft/sec)	$V^2/2g$	Specific Energy
0.403	2.827045	9.112965	3.18354	0.1575	0.560501438
0.404	2.83608	9.123171	3.17339	0.1565	0.560499521
0.405	2.845125	9.133377	3.16331	0.15551	0.56050604

Most computer programs that solve water surface profiles for natural channels use the minimum specific energy approach. For more information, refer to the Bridge Hydaulics Handbook.

2.3.2 Critical Velocity

The velocity at critical depth is called the critical velocity. An equation for determining the critical velocity in an open channel of any cross section is expressed as:

$$v_C = \sqrt{g d_m} \qquad (2\text{-}8)$$

where:

v_C = Critical velocity, in ft/sec

g = Acceleration due to gravity, 32.174 ft/sec^2

d_m = Mean depth of flow, in ft, calculated from

$$d_m = \frac{A}{T} \qquad (2\text{-}9)$$

where:

A = Cross-sectional area, in ft^2

T = Top width of water surface, in ft

2.3.3 Super-Critical Flow

For conditions of uniform flow, the critical depth, or point of minimum specific energy, occurs when the channel slope equals the critical slope (i.e., the normal depth of flow in the channel is critical depth). When channel slopes are steeper than the critical slope and uniform flow exists, the specific energy head is higher than the critical value, due to higher values of the velocity head (kinetic energy). This characteristic of open channel flow is illustrated by the specific head curve segment to the left of critical depth in Figure 2-2 (Part B) and is known as super-critical flow. Super-critical flow is characterized by relatively shallow depths and high velocities, as shown in Figure 2-2 (Part A). If the natural depth of flow in an open channel is super-critical, the depth of flow at any point in the channel may be influenced by an upstream control section. The relationship of super-critical flow to the specific energy curve is shown in Figure 2-2 (Parts A and B).

2.3.4 Sub-Critical Flow

When channel slopes are flatter than the critical slope and uniform flow exists, the specific energy head is higher than the critical value, due to higher values of the normal depth of flow (potential energy). This characteristic of open channel flow is illustrated by the specific head curve segment to the right of critical depth in Figure 2-2 (Part B) and is known as sub-critical flow. Sub-critical flow is characterized by relatively large depths with low velocities, as shown in Figure 2-2 (Part C). If the natural depth of flow in an open channel is sub-critical, the depth of flow at any point in the channel may be influenced by a downstream control section. The relationship of sub-critical flow to the specific energy curve is shown in Figure 2-2 (Parts B and C).

2.3.5 Theoretical Considerations

Several points about Figure 2-2 should be noted. First, at depths of flow near the critical depth for any discharge, a minor change in specific energy will cause a much greater change in depth. Second, the velocity head for any discharge in the sub-critical portion of the specific energy curve in Figure 2-2 (Parts B and C) is relatively small when compared to specific energy. For this sub-critical portion of the specific energy curve, changes in depth of flow are approximately equal to changes in specific energy. Finally, the velocity head for any discharge in the super-critical portion of the specific energy curve increases rapidly as depth decreases. For this super-critical portion of the specific energy curve, changes in depth are associated with much greater changes in specific energy.

2.4 Nonuniform Flow

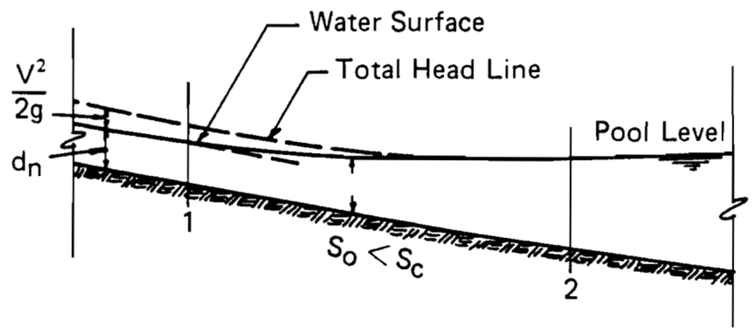
In the vicinity of changes in the channel section or slope that will cause nonuniform flow profiles, the direct solution of Manning's Equation is not possible, since the energy gradient for this situation does not equal the channel slope. Three typical examples of nonuniform flow are illustrated in Figures 2-3 through 2-5. The following sections describe these nonuniform flow profiles and briefly explain how the total head line is used for approximating these water surface profiles in a qualitative manner.

2.4.1 Gradually Varied Flow

A channel on a mild slope (sub-critical) discharging into a reservoir or pool is illustrated in Figure 2-3. The vertical scale is exaggerated for clearer illustration.

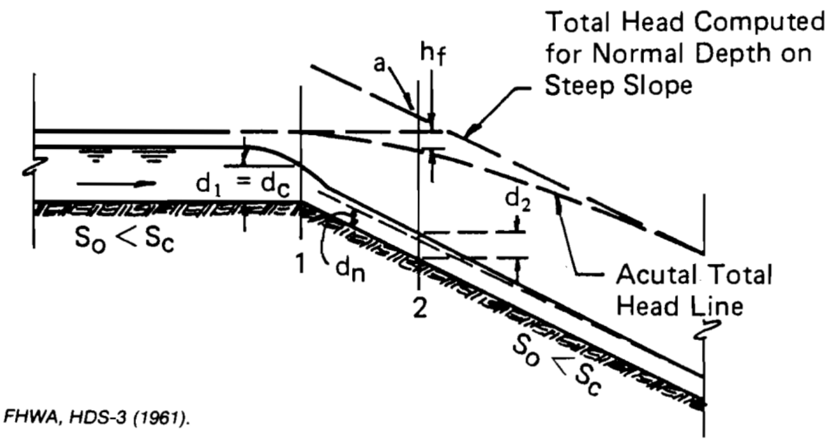
Cross Section 1 is located upstream of the pool, where uniform flow occurs in the channel, and Cross Section 2 is located at the beginning of a level pool. The depth of flow between Sections 1 and 2 is changing, and the flow is nonuniform. The water surface profile between the sections is known as a backwater curve and is characteristically very long.

A channel in which the slope changes from sub-critical (mild) to super-critical (steep) is illustrated in Figure 2-4. The flow profile passes through critical depth near the break in slope (Section 1). This is true whether the upstream slope is mild, as in the sketch, or the water above Section 1 is ponded, as would be the case if Section 1 were the crest of a dam spillway. If, at Section 2, the total head were computed, assuming normal depth on the steep slope, it would plot above the elevation of total head at Section 1 (Point a in Figure 2-4). This is physically impossible, because the total head line must slope downward in the direction of flow. The actual total head line will take the position shown and have a slope approximately equal to S_O, the slope of the channel bottom, at Section 1 and approaching S_O farther downstream. The drop in the total head line (h_{loss}) between Sections 1 and 2 represents the loss in energy due to friction.



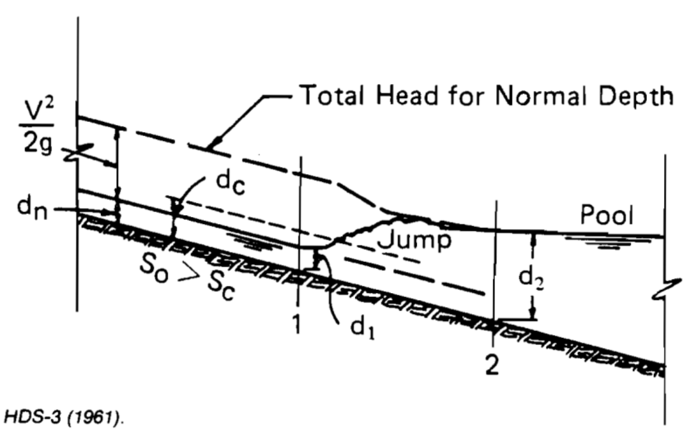
Reference: USDOT, FHWA, HDS-3 (1961).

FIGURE 2-3
Nonuniform Water Surface Profile for Downstream Control Caused by a Flow Restriction



Reference: USDOT, FHWA, HDS-3 (1961).

FIGURE 2-4
Nonuniform Water Surface Profile Caused by a Change in Slope Conditions



Reference: USDOT, FHWA, HDS-3 (1961).

FIGURE 2-5
Nonuniform Water Surface Profile Caused by a Hydraulic Jump

At Section 2, the actual depth (d_2) is greater than normal depth (d_n) because sufficient acceleration has not occurred, and the assumption of normal depth at this point would clearly be in error. As Section 2 is moved downstream, so that the total head for normal depth drops below the pool elevation above Section 1, the actual depth quickly approaches the normal depth for the steep channel. This type of water surface curve (Section 1 to Section 2) is characteristically much shorter than the backwater curve discussed previously.

Another common type of nonuniform flow is the drawdown curve to critical depth that occurs upstream from Section 1 (Figure 2-4) where the water surface passes through critical depth. The depth gradually increases upstream from critical depth to normal depth, provided that the channel remains uniform over a sufficient distance. The length of the drawdown curve is much longer than the curve from critical depth to normal depth in the steep channel.

2.4.2 Gradually Varied Flow Profile Computation

Water surface profiles can be computed using the Energy Equation (Equation 2-2). Given the channel geometry, flow, and the depth at one of the cross sections, the depth at the other cross section can be computed.

The losses between cross sections include friction, expansion, contraction, bend, and other form losses. Expansion, contraction, bend and other form losses will be neglected in the computations presented in this Handbook. Refer to the Bridge Hydraulics Handbook (not yet published) for more information. The remaining loss that must be determined is the friction loss, expressed as:

$$h_f = S_f L \tag{2-10}$$

where:

h_f = Friction head loss, in ft.

S_f = Slope of the Energy Grade Line, in ft./ft.

L = Flow Length between Cross Sections, in ft.

The slope of the Energy Grade Line at each cross section can be calculated by rearranging Manning's equation (Equation 2-4) into the following expression:

$$S = \left(\frac{Qn}{1.49 A R^{2/3}} \right)^2 \tag{2-11}$$

For uniform flow, the slope of the channel bed, the slope of the water surface (hydraulic grade line), and the slope of the energy grade line are all equal. For non-uniform flow, including gradually varied flow, each slope is different.

The slope determined at each cross section can be used to estimate the average slope for the entire flow length between the cross sections. Several different averaging schemes can be used to estimate the average slope, and these techniques are discussed in more detail in the Bridge Hydraulics Handbook (not yet published). The simplest estimate of slope of the energy gradient between two sections is:

$$S_f = \frac{S_1 + S_2}{2} \qquad (2\text{-}12)$$

where:

S_1, S_2 = Slope of the energy gradient at Sections 1 and 2, in ft/ft

The computation of backwater curves in a quantitative manner can be quite complex. If a detailed analysis of backwater curves is required, application of a computer program should be considered. Typical computer programs used for water surface profile computations include HEC-RAS by U.S. Army Corps of Engineers, HEC-2 by the U.S. Army Corps of Engineers (1991), E431 by the USGS (1984), and WSPRO by the USGS (1986). In addition, textbooks by Chow (1959), Henderson (1966), or Streeter (1971), and publications by the USGS (1976b), Brater and King (1976), or the USDA, SCS (NEH-5, 2008) may be useful.

Example 2.4 – Gradually Varied Flow Example

Due to Right-of-Way limitations, the ditch cross section previously used must be reduced to a 3.5 foot bottom width and a 1:3 back slope for a distance of 100 feet. The transition length between the two ditch shapes is 15 feet.

Given: Discharge = 25 ft^3/sec
Roughness = 0.04
Cross Section from Example 2.1
Slope = 0.005 ft/ft

Calculate: Depth of flow in narrower cross section

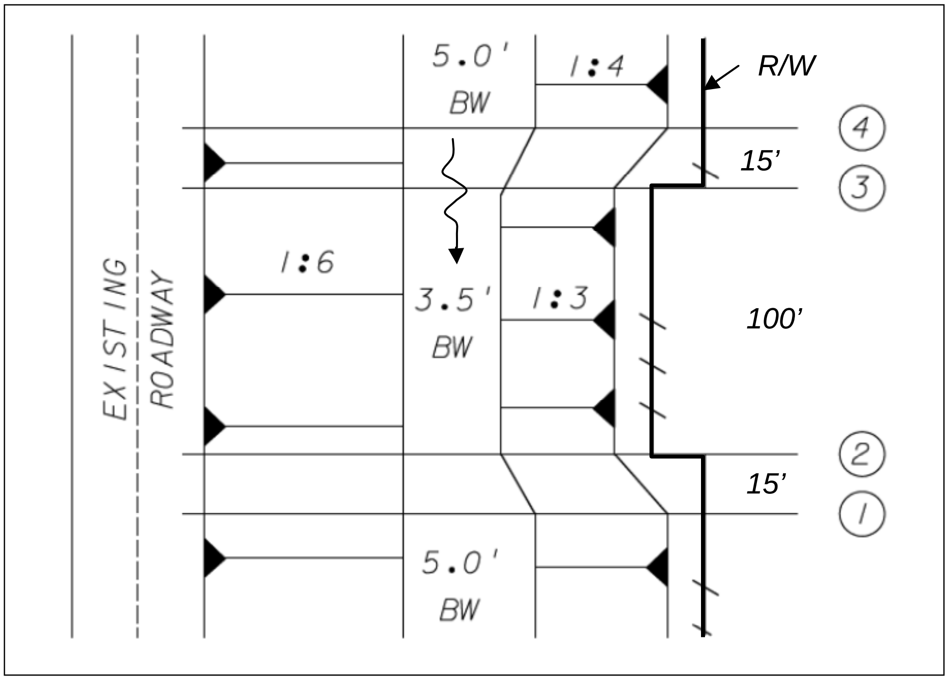


Figure 2-6 Plan View

The flow depths in the two cross sections can be estimated using the slope conveyance method which solves Manning's Equation and assumes that the ditch is flowing at normal depth. Example A.2 shows the computation of the normal depths for the ditch in this problem using the nomographs in Appendix A. The normal depth in the Standard Ditch is 1.12 feet, and the normal depth in the narrowed ditch section is 1.25 feet.

Although it is not standard practice to perform a standard step backwater analysis in a roadside ditch, solving this example will illustrate how a gradually varied profile can be computed using Equations 2-2 and 2-10 through 2-12.

The Froude Number for normal depth flow at the first section is:

$$Area = (1.12 \times 5) + \frac{1}{2}(6 \times 1.12)(1.12) + \frac{1}{2}(4 \times 1.12)(1.12) = 11.87 sq.ft.$$

$$T = 5 + (6+4)1.12 = 16.2\,ft. \qquad D = \frac{A}{T} = \frac{11.87}{16.2} = 0.733$$

$$v = \frac{Q}{A} = \frac{25}{11.87} = 2.11\,fps$$

$$Fr = \frac{v}{(gD)^{1/2}} = \frac{2.11}{(32.174 \times 0.733)^{1/2}} = 0.43$$

Because Fr is less than one, the flow in the channel will be subcritical. Therefore the analysis will start at the downstream cross section and proceed upstream. Assume normal depth in the Standard Ditch at a point just downstream of the downstream transition (Section 1 in the figure above). This assumes that the ditch downstream is uniform for a sufficient distance to establish normal depth at Section 1.

The water depth at Section 1 is 1.12 feet as determined in Example A-2. This depth, along with other geometric and hydraulic values needed for the computations are shown in the first row of the table on the next page. The elevation, z, is arbitrarily taken as zero. Next, the depth at Section 2 will be determined from a trial and error procedure. The first trial depth will be the normal depth at Section 2, which is 1.25 feet. Equations 2-10, 2-11, 2-12, and 2-2 are used to back calculate the depth at Section 2. The back calculated depth of 1.11 feet is shown in the last column. Additional trial depths are assumed until the trial and the back calculated depths agree to the desired accuracy.

Once the depth at Section 2 has been calculated, then the depth at Section 3 can be calculated using the same trial and error process. The same process is also repeated to solve for the depth at Section 4.

	1	2	3	4	5	6	7	8	9	10	11
XS #	Depth Guess (ft)	Area (ft^2)	Perimeter (ft)	Radius (ft)	Velocity (ft /s)	V^2 / 2g (ft)	Z (ft)	EGL (ft)	Slope	Loss (ft)	Depth (ft)
1	1.12	11.872	16.43057	0.72256	2.105795	0.068912	0	1.188912	0.005		
	1.25	11.40625	15.0563	0.75757	2.191781	0.074655	0.075	1.399655	0.00504	0.075301	1.114558
	1.1	9.295	13.66954	0.67998	2.689618	0.112421	0.075	1.287421	0.008766	0.103245	1.104737
	1.104	9.348672	13.70652	0.68206	2.674177	0.111134	0.075	1.290134	0.00863	0.102228	1.105007
2	1.105	9.362113	13.71577	0.68258	2.670337	0.110815	0.075	1.290815	0.008597	0.101977	1.105075
	1.25	11.40625	15.0563	0.75757	2.191781	0.074655	0.575	1.899655	0.00504	0.681854	1.323014
	1.29	12.00345	15.4261	0.77812	2.082735	0.067411	0.575	1.932411	0.004392	0.649423	1.297827
	1.296	12.09427	15.48157	0.78120	2.067094	0.066403	0.575	1.937403	0.004303	0.645002	1.294414
3	1.295	12.07911	15.47233	0.78069	2.069688	0.066569	0.575	1.936569	0.004318	0.645732	1.294977
	1.12	11.872	16.43057	0.72256	2.105795	0.068912	0.65	1.838912	0.004955	0.069549	1.287206
	1.28	14.592	18.06351	0.80781	1.713268	0.045616	0.65	1.975616	0.002827	0.053585	1.294538
	1.294	14.84218	18.20639	0.81522	1.684389	0.044091	0.65	1.988091	0.002699	0.052628	1.295107
4	1.295	14.86013	18.2166	0.81575	1.682355	0.043985	0.65	1.988985	0.002691	0.052562	1.295147

Column 2. Use Area formula for trapezoid from Figure 1-2 and depth guessed in Column 1.
Column 3. Use Wetted Perimeter for trapezoid from Figure 1-2 and depth guessed in Column 1.
Column 4. Column 2 ÷ Column 3
Column 5. Q ÷ Column 2
Column 8. Column 1 + Column 6 + Column 7
Column 9. Solve Equation 2-11 using Column 2 and Column 4 values
Column 10. Calculate S_f with Equation 2-12 using Column 9 from this row and last row of previous section. Calculate the lose with Equation 2-10 by multiplying S_f by the distance to the previous cross section.
Column 11. Back Calculate Depth by Calculating the Total Energy (Col. 8 of previous cross section + Col. 10) and subtracting the Datum and the Velocity Head (Col. 7 + Col. 6).

Looking at the results of the profile analysis on the previous page, several things might not be expected. First, the flow depth at Section 2 (1.105 ft.) is less than the flow depth at Section 1 (1.12 ft.), which might be unexpected because the normal depth of Section 2 is greater than Section 1. However, this is not an unusual occurrence in contracted sections. The reason that the flow depth decreases is because the velocity, and therefore the velocity head, increases. The increase in the velocity head is greater than the losses between the sections; therefore the depth must decrease to balance the energy equation. The opposite can occur in an expanding reach resulting in an unexpected rise in the flow depth even though the normal depth decreases.

The next unusual result is that the flow depth at section 3 is greater than the normal depth in the narrow section. Since the flow depth is less than normal depth at section 2, the water surface profile should approach normal depth from below as the calculations proceed upstream. Therefore, the flow depth at section 3 should be less than the normal depth. The reason that the profile jumps over the normal depth line is because of numerical errors introduced by Equation 2-12. When the change in the energy gradient between two cross sections is too large, Equation 2-12 does not accurately estimate the average energy gradient between the sections. Cross sections must be added between these cross sections to reduce the numerical errors to an acceptable amount. This example has been solved in Appendix C using HEC-RAS. Extra cross sections were included, and the flow depth at section 3 was determined to be 1.24 feet. So, the flow depth had essentially converged to normal depth within the 100 foot distance between sections 2 and 3.

This problem illustrates one of the primary reasons that water surface profiles are not necessary in the typical roadside ditch design. The water depth does not significantly vary from normal depth at any location. So assuming that some freeboard has been included in the design, the ditch will operate adequately when designed by assuming normal depth.

2.4.3 Rapidly Varied Flow

A hydraulic jump occurs as an abrupt transition from supercritical to subcritcal flow. The potential for a hydraulic jump should be considered in all cases where the Froude number is close to 1.0 and/or where the slope of the channel bottom changes abrubtly from steep to mild. For grass lined channels, unless the erosive forces of the hydraulic jump are controlled, serious damage may result.

It is important to know where a hydraulic jump will form, since the turbulent energy released in a jump can cause extensive scour in an unlined channel. For simplicity, it is assumed that the flow is uniform in the channel except in the reach between the jump and the break in the channel slope. The jump may occur in either the steep channel or the mild channel, depending on whether the downstream depth is greater or less than the depth sequent to the upstream depth.

The sequent depth can be calculated using Equation (2-13)

$$d_2 = -\frac{d_1}{2} + \sqrt{\frac{d_1^{\ 2}}{4} + \frac{2v_1 d_1}{g}} \qquad (2\text{-}13)$$

where:

d_2 = Depth below jump, in ft

d_1 = Depth above jump, in ft

v_1 = Velocity above jump, in ft/sec

g = Acceleration due to gravity, 32.174 ft/sec^2

If the downstream depth is greater than the sequent depth, the jump will occur in the steep region. If the downstream depth is lower than the sequent depth, the jump will move into the mild channel. (Chow) For more discussion of the location of hydraulic jumps refer to “Open Channel Hydraulics” by V.T. Chow, PhD.

Once the location of the jump is determined, the length can be determined using Figure 2-8. This figure plots the Froude number of the upstream flow against the dimensionless ratio of jump length to downstream depth. The curve was prepared by V.T. Chow from data gathered by the Bureau of Reclamation for jumps in rectangular channels. It can also be used for approximate results for jumps formed in trapezoidal channels.

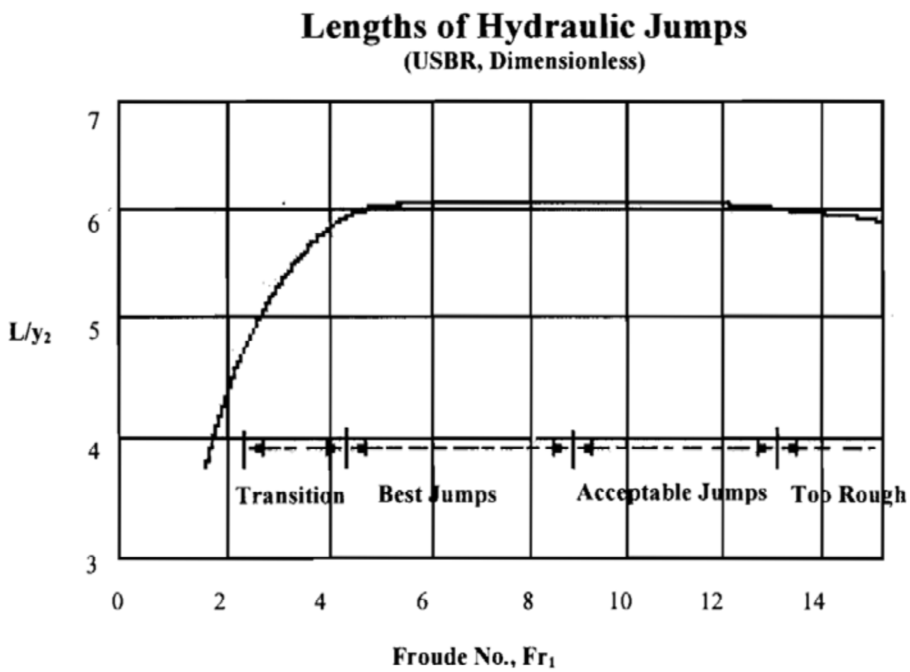


Figure 2-8

Once the location and the length of the hydraulic jump are determined the need for alternative channel lining as well as the limits the alternative lining will need to be applied.

Detailed information on the quantitative evaluation of hydraulic jump conditions in open channels is available in publications by Chow (1959), Henderson (1966), and Streeter (1971), and in HEC-14 from USDOT, FHWA (1983). In addition, handbooks by Brater and King (1976) and the USDA, SCS (NEH-5, 2008) may be useful.

Example 2.5 – Hydraulic Jump Example

Given: $Q = 60.23$ cfs
$V_1 = 13.81$ fps
$g = 32.2\ ft/s^2$
$d_1 = 0.33$ ft
$d_2 = 6.74$ ft

Depths calculated using Manning's equation. The ditch has a 12.5' bottom width with 1:2 side slopes. The longitudinal slopes are 10% and 0.001%, respectively. The roughness value for the proposed rubble riprap is 0.035.

Calculate: Hydraulic Jump and the extent of rubble needed.

Step 1: Calculate Froude number and the Length of the Hydraulic Jump

Froude number, F_1:

$$F_1 = \frac{V_1}{\sqrt{gd_1}}$$

$$F_1 = \frac{13.81}{\sqrt{(32.2)(0.33)}}$$

$$F_1 = 4.24$$

Length of the Hydraulic Jump, L:

From Figure 2-8,

$$\frac{L}{d_2} = 5.85$$

Therefore,

$$L = 5.85 d_2 = (5.85)(6.74)$$
$$L = 39.4\,ft \approx 40\,ft$$

Step 2: Calculate the Upstream Sequent Depth

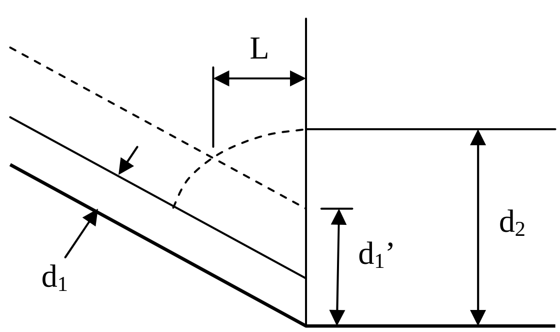
Upstream Sequent Depth, d_1':

$$d_1^{'} = -\frac{d_1}{2} + \sqrt{\frac{2V_1^2 d_1}{g} + \frac{d_1^2}{4}}$$

$$d_1^{'} = -\frac{0.33}{2} + \sqrt{\frac{2(13.81)^2(0.33)}{32.2} + \frac{(0.33)^2}{4}}$$

$$d_1^{'} = 1.81 ft$$

Since the downstream depth d_2 (6.74 ft) is greater than the upstream sequent depth d_1' (1.81 ft), the hydraulic jump occurs in the steep region.



Assuming a more conservative approach, the length of the hydraulic jump should be split between the two regions and rip rap rubble ditch protection should be provided for 20 feet downstream.

2.5 Channel Bends

At channel bends, the water surface elevation increases at the outside of the bend because of the superelevation of the water surface. Additional freeboard is necessary in bends and can be calculated use the following equation:

$$\Delta d = \frac{V^2 T}{g R_C} \qquad (2\text{-}13)$$

where,

Δd = additional freeboard required because of superelevation, ft

V = average channel velocity, ft/s

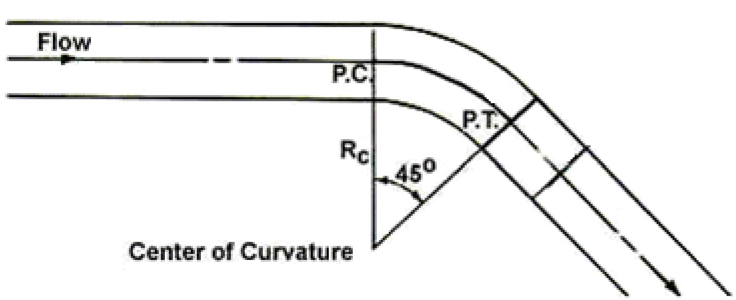
T = water surface top width, ft

g = acceleration due to gravity, ft/s^2

R_C = radius of curvature of the bend to the channel centerline, ft

Example 2.5 – Channel Bend Example

The channel of Example 2.2 takes a 45 degree bend with a radius of 30 feet. What is the increased depth on the outside of channel at the bend?



From Example 2.2, V = 1.192 ft/sec

Top Width

$$T = 5 + 0.826(4 + 6) = 13.26\,ft.$$

$$\Delta d = \frac{V^2 T}{g R_C} = \frac{1.192^2 (13.26)}{32.174(30)} = 0.02\,ft$$

The depth of flow on the outside edge of the ditch is 0.86 + 0.02 = 0.88 ft.

The superelevation is insignificant for this example problem, as it is for many ditches in Florida. The variable that affects water surface superelevation the most is the velocity because it is squared in Equation 2-13. Ditches with a high velocity at a bend with a small radius will have greater superelevations.

Chapter 3

Open Channel Design

The channel shape, slope and roughness were given in the example problems of the previous chapters. The flow depths and velocities were then determined using the analysis methods described in these chapters. If a project incorporates existing channels, then the analysis methods can be applied to those channels just as they were applied to the example problems. However, many projects will require new channels that must be designed. This chapter discusses how the drainage designer selects the channel geometry and channel linings for FDOT projects.

3.1 Types of Open Channels for Highways

Open channels can be generally classified as those which occur naturally and those which are manmade, including improved natural channels. The latter, called artificial channels are used on most roadway projects. The types of channels commonly used on FDOT projects are listed in Chapter 2 of the FDOT Drainage Manual:

- Roadside Ditch
- Median Ditch
- Interceptor Ditch
- Outfall Ditch
- Canals

Design frequencies are recommended for each of these channel types in Section 2.2 of the Drainage Manual.

The roadside ditch receives runoff from the roadway pavement and shoulders as directed by the cross slope and shoulder slopes. The roadside ditch may also receive flow from offsite drainage areas on adjacent properties. The ditch may also intercept ground water to protect the base of the roadway. The roadside ditch conveys the flow to an outfall point, although the ditch may flow into other ditches or components of the stormwater management system before reaching the ultimate outfall point from FDOT Right of Way. Depressed medians will collect runoff and a median ditch will be needed to convey runoff to an outfall point. In general, roadside and median ditches are relatively shallow trapezoidal channels, while swales are shallow triangular, zero bottom width channels.

Interceptor ditches can have various purposes. Interceptor ditches can provide a method for intercepting offsite flow above cut slopes, thereby controlling slope erosion. An interceptor ditch can also be used to collect offsite flow and keep it separate from the project stormwater. This flow can bypass the stormwater treatment facilities, reducing their size and cost.

Outfall ditches are designed in most cases to receive runoff from numerous secondary drainage facilities, such as roadside ditches or storm drains. The delineation between a roadside ditch and an outfall ditch can become blurred. If the discharge from a stormwater management facility is brought back to the roadside ditch to convey the flow to another point on the project for ultimate discharge, then the roadside ditch should be considered an outfall for the purpose of selecting the design frequency. If considerable flows from offsite areas and onsite project flows are combined together in the roadside ditch to become a significant discharge, then the roadside ditch should be considered an outfall for the purpose of selecting the design frequency. The use of a roadside ditch as an outfall ditch is not recommended, since its probable depth and size could create a potential hazard.

Canals, like outfalls, are also large artificial channels that accept flows from other drainage components. The added connotation of a canal is that there is always water in the channel, unlike many outfalls that only flow immediately after a rainfall event. If the canal, which always has water, is close to the road, then it can be a potential hazard. For the purpose of identifying a hazard, the FDOT Plans Preparation Manual defines a canal as an open ditch parallel to the roadway for a minimum distance of 1000 ft., and with a seasonal water depth in excess of 3 ft. for extended periods of time (24 hours or more). Water Management Districts and Local Agencies may have a different definition for canals when determining regulatory jurisdiction.

Other types of ditches are mentioned in other FDOT publications. Right of Way ditches are mentioned in the Specifications for Road and Bridge Construction and a detail is given on Index 281 of the Design Standards. The Right of Way ditch often functions as a type of relief ditch, handling drainage needs other than those for the roadway and thus freeing roadside ditches from carrying anything except roadway runoff. Right of Way ditches can usually be considered interceptor ditches when selecting the design frequency.

The term ‘lateral ditch’ is used in the FDOT Plans Preparation Manual and the FDOT Standard Specifications for Road and Bridge Construction. Two reasons that the term is used are to:

- Determine how the ditch excavation will be paid.
- Determine how the ditch is shown in the plans.

A lateral ditch is generally perpendicular to the roadway and can either flow to or away from the road. However, a lateral ditch can also be parallel to the road right of way if the ditch or channel is separate from the roadway template. Refer to the Plans Preparation Manual for guidance on selecting the excavation pay item. Consider the purpose of the lateral ditch and fit it to one of the ditch types listed above to select the design frequency.

Several FDOT publications use the term roadway ditch rather than roadside ditch. These two terms are interchangeable. Many other terms referring to open channels are used in other FDOT publications or by engineers performing work for the Department. The definitions of most of these terms are self explanatory because of their descriptive names. Some examples are:

- Drainage Ditch
- Stormwater Ditch
- Bypass Ditch
- Diversion Ditch
- Conveyance Channel
- Agricultural Ditch
- Treatment Swale

A swale is a special kind of artificial ditch which has become important in Florida. The following legal definition of a swale as it relates to the regulation and treatment of stormwater discharge is from Chapter 62-25.020, FAC:

"Swale" means a manmade trench which:

a) has a top width-to-depth ratio of the cross section equal to or greater than 6:1, or side slopes equal to or greater than 3 feet horizontal to one-foot vertical; and

b) contains contiguous area of standing or flowing water only following a rainfall event; and

c) is planted with or has stabilized vegetation suitable for soil stabilization, stormwater treatment, and nutrient uptake; and

d) is designed to take into account the soil erodibility, soil percolation, slope, slope length, and drainage area so as to prevent erosion and reduce pollutant concentration of any discharge.

3.2 Roadside Ditches

The following steps may be followed to design a roadside ditch.

Step 1 – Establish a Preliminary Drainage Plan. The roadside ditches will be components of an overall drainage system. Since the roadside ditch will generally follow the grade of the road, the high points in the roadway grade will be initial drainage boundaries. However, these boundaries can be adjusted by using special ditch grades so that the ditch flows in a different direction than the roadway grade. The boundaries can be adjusted significantly for projects in flat terrain. It is best to keep existing drainage patterns if possible. Low points can also be adjusted with special ditch grades if the desired discharge point is not at the low point of the roadway grade.

Most projects will have stormwater management facilities, so the roadside ditches will connect with the conveyance components to the various facilities. Not all portions of the roadside ditch can physically be directed to a stormwater management facility, so short segments may need to discharge to other points such as streams or ditches near cross drains and bridges, or other points along the roadway.

When initial ditch grades are determined, a ditch slope with sufficient grade to minimize ponding and sediment accumulation should be provided. The Drainage Manual requires a minimum physical slope of 0.0005 ft/ft to be used for ditches where positive flow is required. These flat slopes are difficult to grade during construction and the flow is easily impeded by clumps of grass left behind by mowers.

Existing utilities may also control the grade of the ditch because minimum cover over the utility must be maintained.

Step 2 – Select Standard Ditch components. The standard roadside ditch will be shown in the Plans on the Typical Section. Standard Ditch sections are given in the Plans Preparation Manual for several roadway types, and are shown in the Figures 3-1 and 3-2 below. The Standard Ditch may need to be adjusted due to peculiarities that are consistent throughout the project. An example might be a narrow border width and limited Right of Way.

The typical ditch shown in Figure 3-1 for 2 lane roads is narrower than most mitered end sections. In some situations, a wider typical ditch section might be used. If the wider ditch is not used, then the right of way at each mitered end should be checked to be sure the right of way will be adequate to accommodate a wider ditch at the mitered end section.

If the ditch size needs to be reduce due to right of way limitations, the following may be considered to reduce the ditch size:

- The front slope must remain 1:6 in clear zone, but can break over to 1:4 at the clear zone

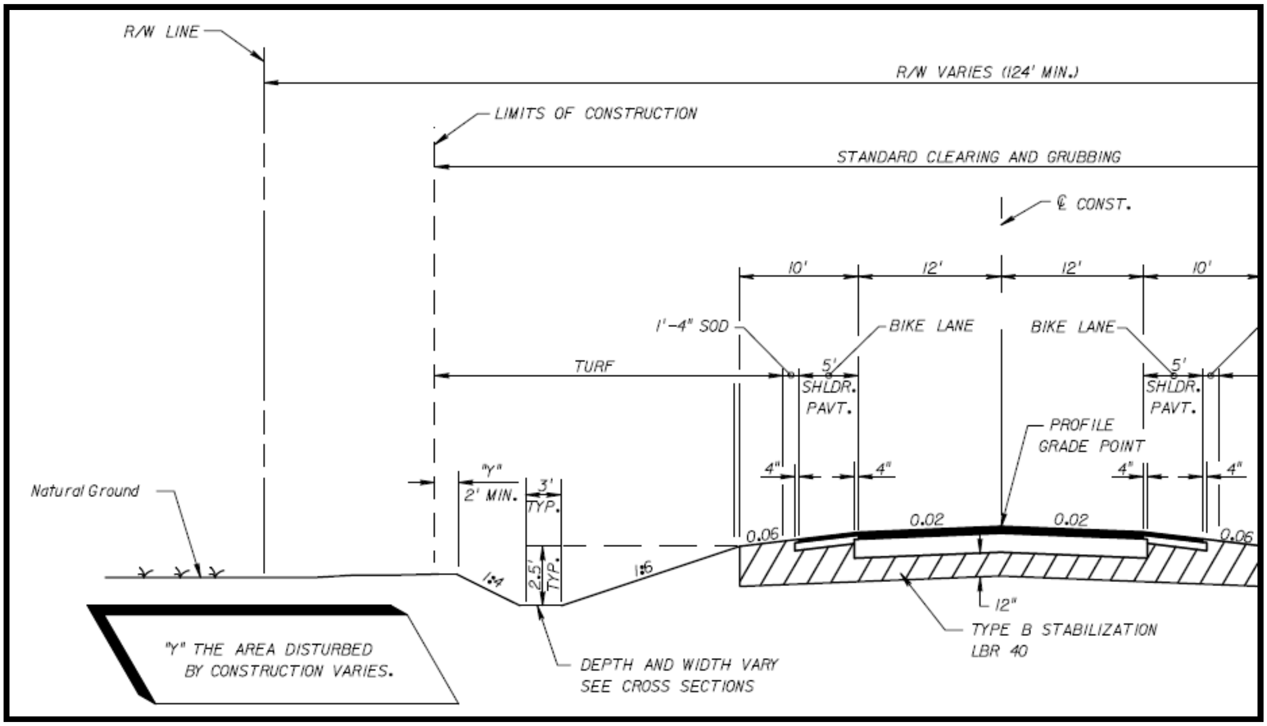


Figure 3-1 Typical Ditch for Two Lane Rural Roadway

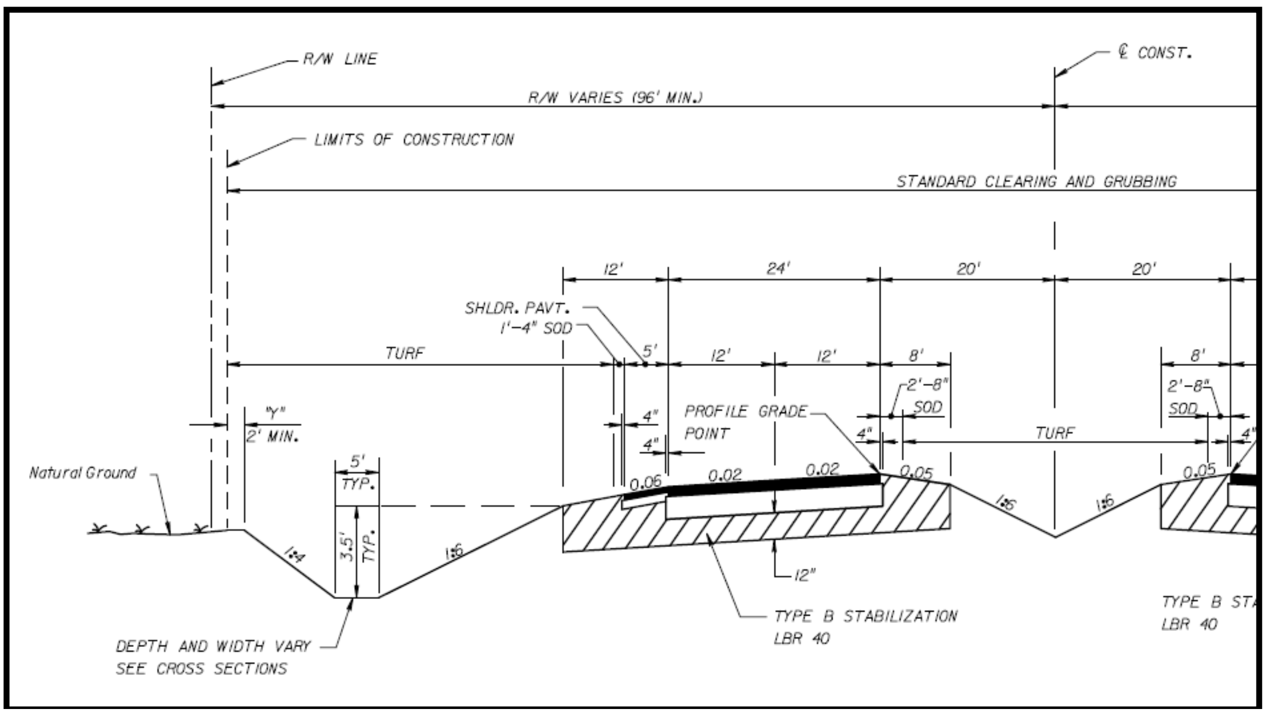


Figure 3-2 Typical Ditch Roadside and Median Ditches

- The bottom width can be narrowed. Three feet is a desirable minimum, but Maintenance and Construction may have equipment to build and maintain a two foot bottom width. V bottomed ditches with steep side slopes should be avoided, if possible. The bottom width should not be narrower than the sidedrain endwalls.

- The back slope can be steepened if the following is considered:
  - Steeper slopes are harder to maintain, especially 1:3 and steeper
  - Check the soils for stability
  - Significant offsite drainage down a steep back slope will cause erosion on the slope

- The depth to the shoulder point can be reduced if the following is considered:
  - Check the ditch capacity
  - Consider type of facility and base clearance needs

The ditch can also be enclosed with a pipe system, although a ditch or swale is still usually needed to collect the roadway runoff into inlets. Enclosing the system will increase construction costs, but may be less expensive than obtaining more Right of Way.

Step 3 – Check for locations where the Standard Ditch will not work. A good way to check is to plot the standard ditch on the cross sections. Look for places where the ditch extends beyond the Right of Way or conflicts with utilities and other obstructions. Also look in the Plan View to check for obstructions between the cross sections.

The size of the ditch can be adjusted considering the same issues identified in the previous step. If the grade of the ditch must be adjusted, then a special ditch profile must be developed and plotted in the plans. Some locations where the ditch grade may need to be adjusted include:

- Outfall locations – the grade of the Standard Ditch will follow the grade of the road. If the outfall location is not at the lowest point in the roadway profile, then a special ditch profile is needed.
- Locations of high water table. Note: these areas may require feedback to the roadway designer to raise the roadway grade.
- Structures such as cross drains, median drains, and side drains may need to be at a lower elevation than the standard ditch elevation. If the entrance end of the culvert is depressed below the stream bed, more head can be exerted on the inlet for the same headwater elevation. Usually the sump is paved, but for small depressions, an unpaved excavation may be adequate.
- Locations where top of back slope creates a ditch that is too shallow. Sometimes a berm can be used to contain the ditch instead of changing the grade. Be careful that offsite drainage is not blocked. If a berm is used, provide an adequate top width and side slopes for ease of maintenance. A suggested minimum top width is 3 feet, but 5 feet is desirable.

Special ditch profiles will be needed if the profile grade is less than the minimum ditch slope. Refer to the Drainage Manual for minimum ditch slope criteria. At vertical curve crests the ditch grade will be less than minimum ditch grade criteria given in the Drainage Manual. (In fact the ditch grade will go to zero at the high point.) A special ditch grade is not necessary at a vertical curve crest.

Step 4 – Compute the Flow Depths and Velocities. Although some designers check the ditch at regular intervals, it is not necessary. Checking at critical locations is adequate. Check the ditch at the outfall point. The discharge will be greatest at this location, so it may represent the worst case conditions for the entire ditch. Other critical locations that should be checked are:

- Changes in slope, specifically steeper slopes
- Changes in shape, specifically narrower sections
- Shallowest ditch depths
- Changes in lining (roughness)
- Changes in flow

Determine the maximum allowable depth of the ditch at these sections including freeboard. At a minimum, freeboard should be sufficient to prevent superelevation changes or fluctuations in the water surface from overflowing the sides. The construction tolerances given in the Specifications for Road and Bridge Construction should also be included when determining the freeboard. Typically 0.5 feet of freeboard is considered adequate. The actual depth of flow in the ditch needs to be checked against the allowable depth at these locations to ensure that the stormwater in the ditch does not overtop the sides and discharge to adjacent properties. The depth of flow in the ditch may also cause a backwater effect to cause flooding and thus damage to the adjacent properties.

If the actual depth exceeds the maximum allowable depth in the ditch then the ditch does not have enough capacity. Possible considerations for increasing the capacity are:

- Increase bottom width
- Make ditch side slopes flatter
- Make longitudinal ditch slope steeper
- Provide a smoother ditch lining
- Install drop inlets and a storm drain pipe beneath the ditch
- Berm up the back slope of the ditch

Step 5 - Check Lining requirements. Once the ditch geometry components are set and the depth of flow is determined adequate then the ditch needs to be checked to determine if a ditch lining is required. The maximum velocity in the ditch should be checked against the allowable velocities for bare earth shown in Table 2.3 of the

Drainage Manual. If these velocities are met then the standard treatment of grassing and mulching should be used.

If the maximum ditch velocity exceeds the allowable velocity for bare earth then sodding, ditch paving, or other form of ditch lining should be provided. See Chapter 4 for more discussion of ditch linings.

3.3 Median Ditches

The design steps are similar to those for roadside ditches:

Step 1 – Establish a Preliminary Drainage Plan. The median ditches will also be components of an overall drainage system. The grade of the median ditch will also generally follow the grade of the road. Generally, curbs are not provided on the edge of pavement and the median ditch drains part or all of the shoulder area in addition to the median itself. Even where curbs are provided, it is preferable to slope medians wider than 15 feet to a ditch. This keeps water in the median off the pavement. Medians less than 15 feet wide are generally crowned for drainage and if they are less than 6 feet in width they are usually paved. Permitting agencies may request that the median ditch be depressed.

Once the width of median ditch is established, locate outfall points from the median. If the travel lanes slope to the outside and the median is impervious, then the median runoff may not need to be conveyed to a stormwater treatment facility. The median may be able to discharge directly into cross drains via inlets.

Continuous flow in medians is often interrupted by median cross overs, bridge piers, or other structures. Decide whether to convey around the obstruction or to one side of the roadway. Consider the flow depth in the median, feasible means to convey around the obstruction, the size of pipe to convey to the outside, the cover available, and the elevation of the roadside ditch to which the flow will be conveyed. Also consider the actual low point of the median ditch, which is usually at the low point of the roadway grade. This may be affected by guardrail, turn lanes, etc. Turn lanes and other non-typical roadway configurations may also create a depressed gore area. These areas will also need to be analyzed with similar methods as those used for median ditches.

Considerations to determine which side of the roadside to discharge to include:

- Maintenance of Traffic phasing and construction sequencing
- Which side the outfall or stormwater facility is located
- Comingling with offsite runoff

Step 2 – Select Standard Ditch components. The standard median ditch will be shown in the Plans on the Typical Section. Standard ditch sections are given in the Plans Preparation for several roadway types, and one is shown in the Figure 3-2.

Step 3 – Compute the Flow Depths and Velocities. Determine critical locations to check depth of flow and velocities as outlined above. In addition to the critical areas for the roadside ditch, the median ditch should also be evaluated in gore areas caused by turn lanes or additional pavement. If the actual depth exceeds the maximum allowable depth then the capacity of the ditch will need to be increased. Methods similar to increasing the capacity of a roadside ditch should be used. The designer should be mindful of the additional clear zone requirements for median ditches.

Step 4 - Check Lining requirements. Once the section of the ditch is established, the maximum velocities need to be checked against the allowable velocities for bare soil. If those velocities are exceeded, then additional evaluation will be required to determine the appropriate lining for the ditch. See the documentation in Chapter 4 for further discussion.

3.4 Interceptor Ditches

Interceptor ditches are located along the natural ground near the top edge of a cut slope or along the edge of the right of way to intercept the runoff before it reaches the roadway. Interceptor ditches along the edge of the right of way are commonly referred to as right of way ditches.

The interceptor ditch will generally follow the grade of the natural ground adjacent to the project, not the profile grade of the road. The high points in an interceptor ditch should be located at the drainage divides of the adjacent property to keep existing drainage patterns if possible. Low points will also typically follow the adjacent terrain allowing the interceptor ditch to discharge to points such as streams near cross drains and bridges.

Most projects will have stormwater management facilities. These facilities are often offset from the project area so it is important to consider conflicts that may arise where the outfall ditch intersects the interceptor ditch.

The design steps for interceptor ditches are the same as those of the roadside ditch. See Section 3.2 for the design procedure.

3.5 Outfall Ditches

Since outfall ditches are receptors of runoff from numerous secondary drainage facilities, including stormwater management facilities, the standard ditch section needs to be designed for a larger capacity. The standard ditch section needs to be evaluated against the clear zone criteria for the project. Even though outfall ditches have a larger design event and carry larger flows, the design steps are the same as those of the roadside ditch. See Section 3.2 for the design procedure.

The design should also include consideration for the following:

- The drainage area flowing into the outfall ditch by overland flow. Designers often forget to include this area in the total drainage area when determining the design

flow rates for the outfall ditch. Another concern is erosion down the side slope from the sheet flow from these areas. Spoil from the ditch construction could be use to create berms to block and collect the flow in inlets to prevent this erosion.
- Check for existing outfall easements. Some easements may require a specific type of conveyance, such as a ditch or a pipe system.

3.6 Hydrology

As stated in Section 2.3 of the Drainage Manual, hydrologic data used for the design of open channels shall be based on one of the following methods as appropriate for the particular site:

- A frequency analysis of observed (gage) data shall be used when available
- Regional or local regression equation developed by the USGS
- Rational Equation for drainage areas up to 600 acres
- For an outfall from a stormwater management facility, the method used for the design of the stormwater management facility may be used
- For regulated or controlled canals, hydrologic data shall be requested from the controlling entity

For a more detailed discussion on procedure selection and method for calculating runoff rates refer to the Hydrology Drainage Handbook.

3.6.1 Frequency

Roadside or median ditches or swales, including bypass and interceptor ditches, are usually designed to convey a 10-year frequency storm without damage; outfall ditches or canals should convey a 25-year frequency storm without damage. However, because the risks and drainage requirements for each project are unique, site-specific factors may warrant the use of an atypical design frequency. Regardless of the frequency selected, the potential for flooding that exceeds standard criteria should always be considered. Pre-development stages for all frequencies up to and including the 100-year event must not be exceeded unless flood rights are obtained or the flow is contained within the ditch.

It is also important to consider sediment transport requirements for conditions of flow below the design frequency. A low flow channel component within a larger channel can reduce the maintenance effort by improving sediment transport in the channel.

Temporary open channel facilities used during construction should be designed to handle flood flows commensurate with risks. The recommended minimum frequency for temporary facilities and the temporary lining of permanent facilities is 20 percent of the standard frequency for permanent facilities, which extrapolates as a 2-year frequency for roadside ditches and a 5-year frequency for outfall ditches.

3.6.2 Time of Concentration

The time of concentration is defined as the time it takes runoff to travel from the most remote point in the watershed to the point of interest. When using the Velocity Method, the time of travel for main channel flow is calculated using the velocity in the section and the channel length. Segments used to determine the velocity should have uniform characteristics. A new segment should be used each time there is a change in the channel geometry such as cross section or channel slope. The time is calculated for each segment and then added together to determine the total time of concentration for the channel. Methods and procedures to determine the time of concentration are discussed in the FDOT Hydrology Handbook.

3.7 Tailwater and Backwater

The water depth at the downstream end of the ditch will affect the flow depth and velocities in the ditch for some distance upstream. The downstream water depth, or tailwater, may cause a backwater condition with a gradually varied water surface profile. In roadside ditches, the water surface profile can be approximated as a flat water surface at the tailwater (T_w) elevation that intercepts the normal depth (d_n) of flow in the ditch as shown in Figure 3-3. If the tailwater depth is less than the normal depth in the ditch, then the water surface profile in the ditch should be approximated as the normal depth in the ditch as shown in Figure 3-4. For the low tailwater condition, the velocity check for lining requirements should be done using the velocity for the tailwater depth, not the normal depth.

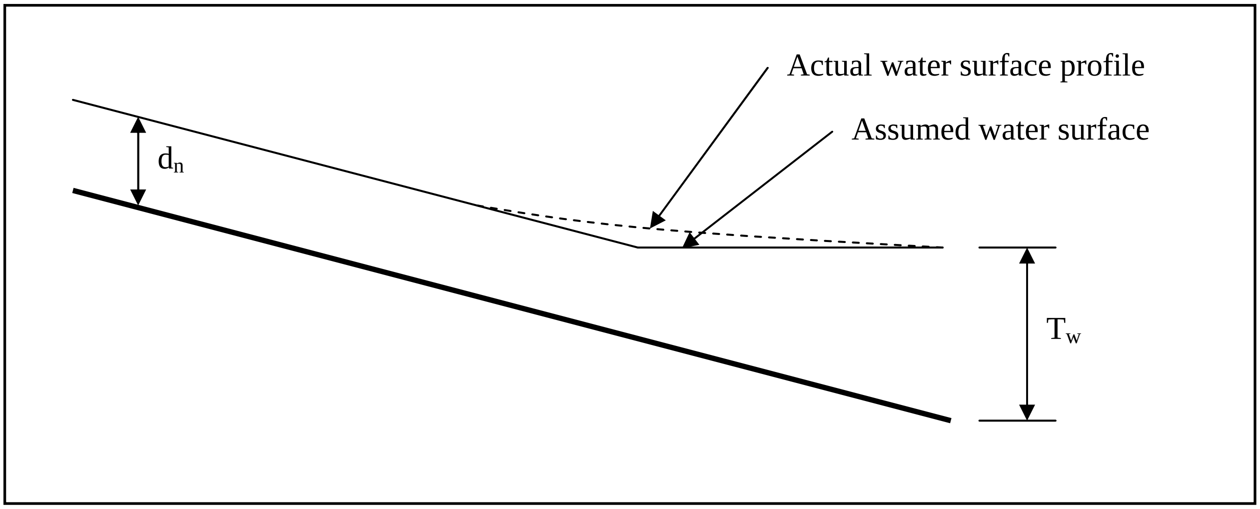


Figure 3-3 Assumed Water Surface for $T_w > d_n$

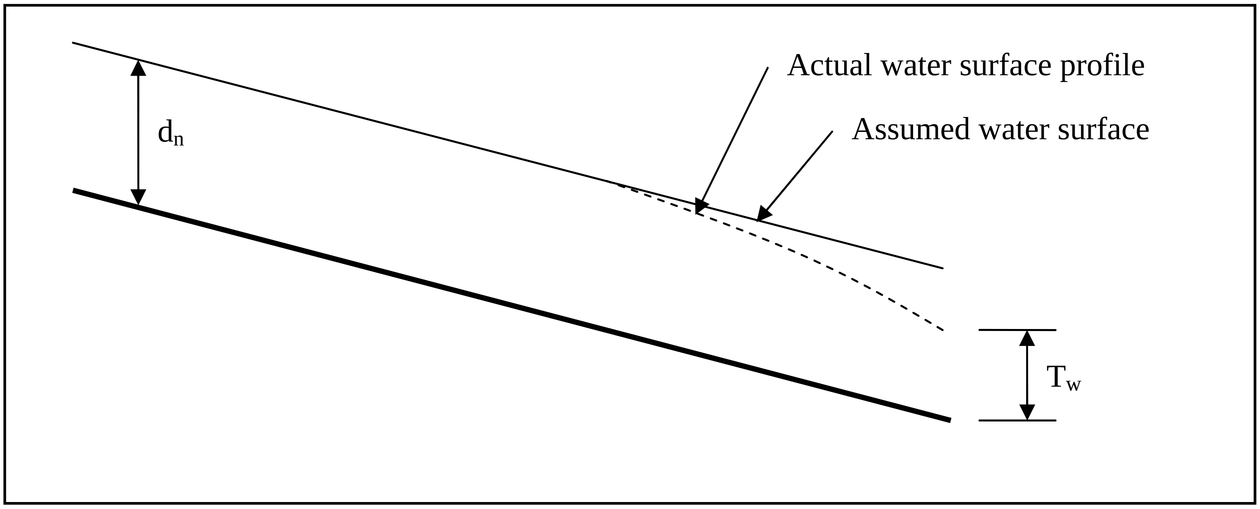


Figure 3-4 Assumed Water Surface for $T_w < d_n$

To summarize the water surface approximation, the water surface elevation at any point in the ditch is the higher of the normal depth elevation or the tailwater elevation. The frequency of the design tailwater elevation can be determined using the same recommendations for storm drains in Section 3.4 of the Drainage Manual.

The same water surface profile assumptions illustrated above also apply to other backwater conditions in the ditch. Side drains are an example. The water surface elevation in the ditch at any point upstream of a side drain should be the greater of the normal depth elevation or the headwater elevation of the culvert. The normal depth in the ditch changes if the ditch slope, cross section or roughness changes. If the downstream normal depth is greater, then the assumed water surface is shown in Figure 3-5.

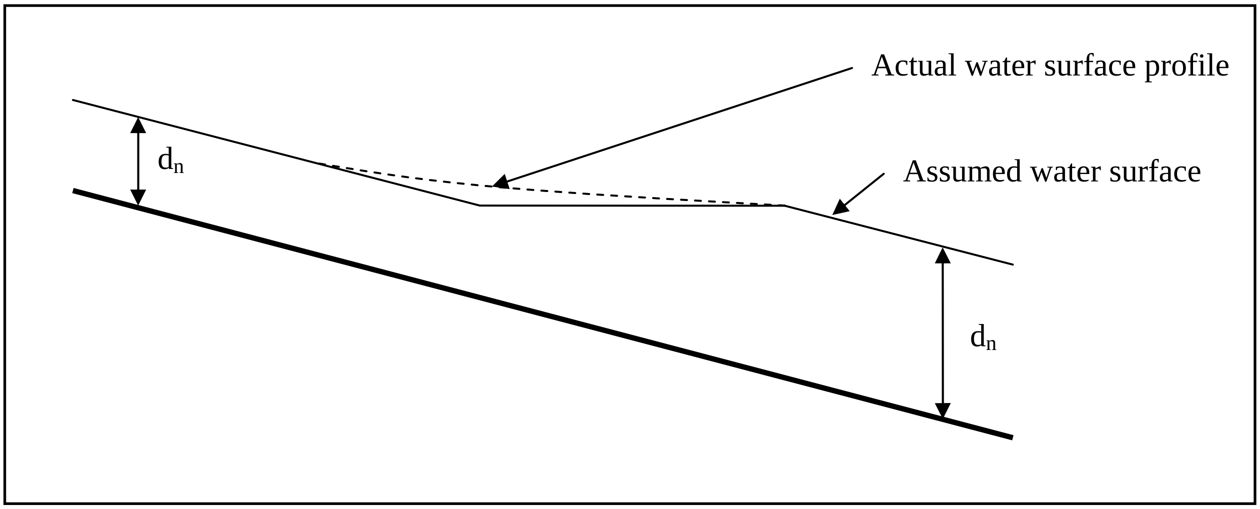
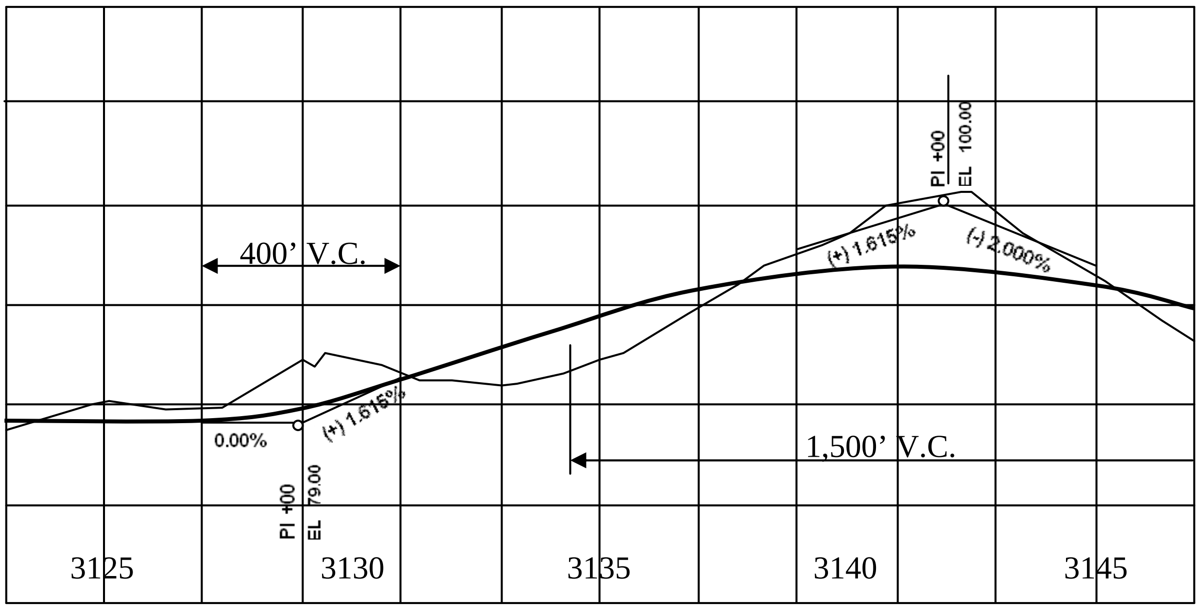
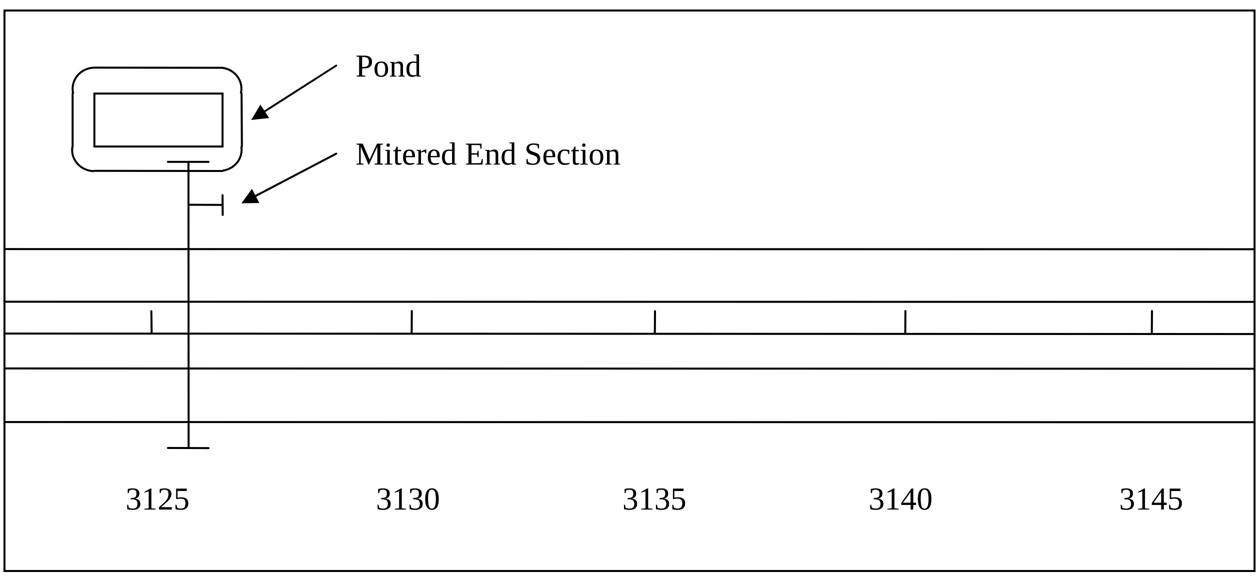


Figure 3-5 Assumed Water Surface for change in d_n

Example 3.1 – Roadside Ditch Design Example

Plan and profile views of a proposed four lane roadway are shown in the figures below. Complete the design of the left roadside ditch.





Step 1 – Drainage Plan. A stormwater pond to treat and attenuate the roadway runoff has been located on the left side of the roadway near Station 3125+00. Roadside ditches will collect the runoff from the roadway and convey it to the cross drain, which empties into the pond. The offsite drainage area is small; therefore, dual ditches are not needed to reduce the size of the pond.

The left roadside ditch will discharge into a mitered end section at Station 3126+50. The design frequency for the ditch will be 10 years (refer to the Drainage Manual for the design frequency). The pipe system and the pond may have different design frequencies than the ditch, but a 10 year elevation in the pond and the 10 year hydraulic grade line for the pipe system at the mitered end section can be determined. The hydraulic grade line of the pipe system at this headwall will be the tailwater elevation for the ditch.

The design of the overall drainage system may be iterative. The design of one component, such as the pond, can affect the design of other components such as the left and right roadside ditches, the cross drain, and even the median ditch. To simplify this example, the tailwater elevation for the ditch will be given as 76.52.

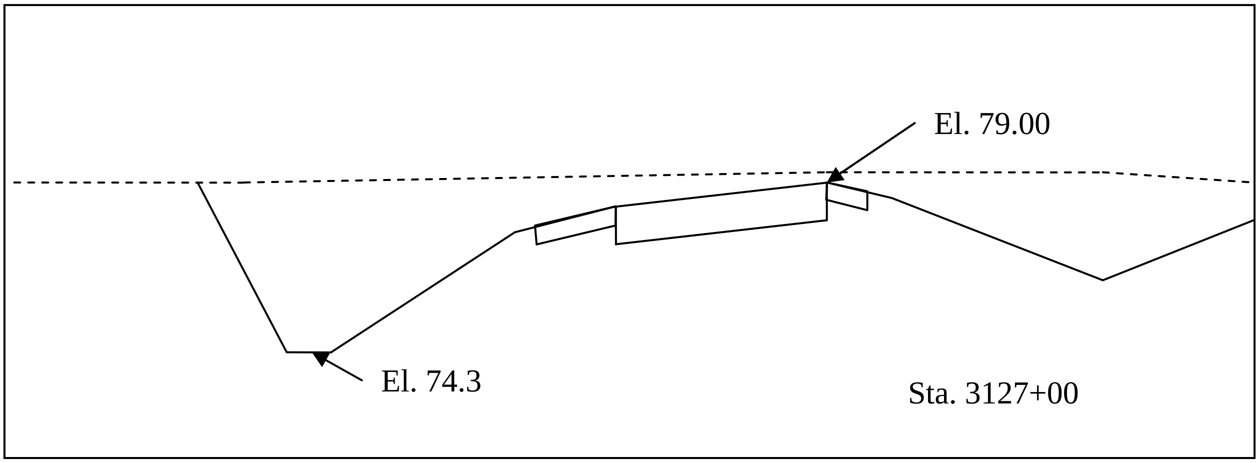
Step 2 – Standard Ditch Components. The standard ditch shown in Figure 3-2 will be used. The vertical distance from the profile grade line (PGL) to the ditch bottom elevation of the Standard Ditch will be:

Elevation Difference = (24 ft. x 0.02) + (12 ft. x 0.06) + 3.5 ft. = 4.7 ft.

Step 3 – Check for locations where the Standard Ditch will not work. Three reasons why the standard ditch will not work are:

- The backslope tie in to natural ground extends beyond the right of way line and acquiring additional right of way is not prudent.
- The natural ground elevation is lower than the standard ditch bottom elevation, or low enough that the standard ditch is too shallow.
- The profile grade is less than the minimum ditch slope.

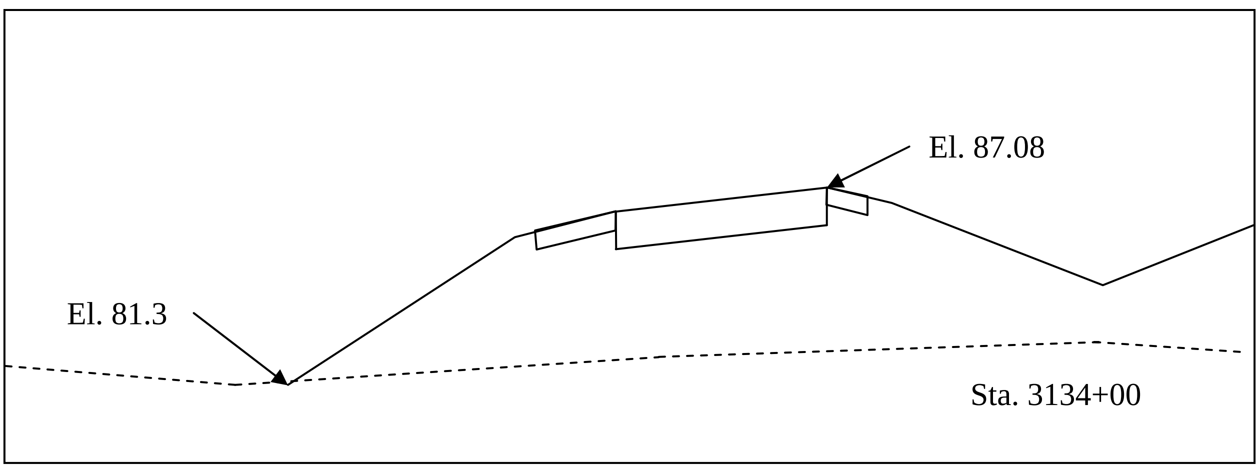
Plotting the standard ditch on the roadway cross sections is a good way to look for locations where the Standard Ditch will not work. Also, starting at the downstream end of the ditch and working upstream will afford an orderly approach to design the ditch. For this example, the profile grade elevation will be 79.00 and the bottom of the standard ditch will be 74.3 at Station 3127+00, as shown in the figure below.



The PGL is flat (0.000%) between this cross section and the end section at Station 3126+50. The minimum slope of the ditch 0.05%, and the desirable slope is at least 0.1%. Therefore, a special ditch grade will be needed between these stations. If the flowline at the headwall (Station 3126+50) is set at 74.2, the ditch grade between these stations will be 0.1 / 50 = 0.002, or 0.2%.

At this point in the design process, the discharge at the downstream end of ditch would be calculated. For this example, the discharge will be given as 12.7 cfs at the end section. Refer to the Hydrology Handbook for an explanation of how to calculate the discharge. Solving Manning’s Equation with the standard ditch shape (5’ bottom width, 1:6 front slope, 1:4 back slope), the slope of 0.2%, n = 0.042, and the discharge of 12.7 cfs gives a flow depth in the ditch of 1.03’. At the headwall, the normal depth elevation would be 74.2 + 1.03 = 75.23. This elevation is less than the tailwater elevation. Therefore, the flow depth in the ditch is the tailwater elevation of 76.52. The outside edge of the shoulder elevation is lower than the back of ditch elevation at this location, and will therefore control the allowable flow depth in the ditch. Since the tailwater elevation is lower than the allowable flow depth the ditch depth is adequate.

Proceed upstream to continue the design. Looking at the cross sections between Stations 3133+00 and 3136+00, the standard ditch bottom elevation will be higher than the natural ground elevation for several hundred feet, as typified by the cross section shown below for Station 3134+00.



The standard ditch could be used if a berm was constructed. However, there are at least two reasons not to construct the berm. First, some offsite flow to the ditch would be blocked. Second, the cost of constructing the berm is unnecessary since a special ditch profile can be used to lower the ditch into the natural ground.

The discharge needs to be determined at this point to continue the design. A conservative assumption would be to use the discharge at the downstream end of the ditch. In this case, the designer judges that the discharge might be significantly different and calculates the discharge at this point. To simplify the example, the discharge at this location is given as 10.2 cfs.

Assuming a ditch bottom elevation of about 79.3 (2 feet below natural ground), the slope to Station 3127+00 would be (79.3 – 74.3) / 500 = 0.001, or 0.1%. Selecting the value of 2’ was based on some preliminary calculations of the flow depth and including some freeboard. Solving Manning’s Equation with the standard ditch shape, the slope of 0.1%, n = 0.042, and the discharge of 10.2 cfs gives a flow depth in the ditch of 1.10’. This would leave a freeboard of 0.9’ at this location, which is more than needed. Therefore, a special ditch grade of 0.1% will be used between Stations 3127+00 and 3134+00.

The flow depth must be checked at Station 3127+00 with the flatter slope of 0.1%. The discharge at 3126+50 will be used to be conservative. Solving Manning’s Equation with the standard ditch shape, the slope of 0.1%, n = 0.042, and the discharge of 12.7 cfs gives a flow depth in the ditch of 1.22’. The normal depth at Station 3127+00 would be 74.3 +1.22 = 75.52. This is less than the tailwater, so the flow depth would be 76.52. Therefore, the ditch will still contain the flow with the flatter slope. Checking the cross sections between 3127+00 and 3134+00, the ditch would not be shallower than 2’.

The special ditch grade has to tie back into the standard ditch grade someplace further upstream. The standard ditch bottom will return to an adequate depth into natural ground to contain the flow at Station 3137+00. The PGL at Station 3137+00 is 91.17. The ditch bottom elevation for the standard ditch is 86.47. The ditch grade will be (86.47 – 79.3) / 300 = 0.0239, or 2.39%. Solving Manning’s Equation with the Standard Ditch shape, the slope of 2.39%, n = 0.06, and the discharge of 10.2 cfs gives a flow depth in the ditch of 0.59’ and a velocity of 2.2 fps. Note that the roughness changes because the flow depth is less than 0.7 feet. The velocity is low enough that ditch lining will not be needed. However, sod will be needed, instead of seed and mulch, to establish grass during construction.

Checking the cross sections between 3134+00 and 3137+00, the ditch depth is at least 1.5’, which will provide acceptable freeboard.

To summarize, the special ditch grades will be:

- 0.2% from Station 3126+50 to 3127+00
- 0.1% from Station 3127+00 to 3134+00
- 2.39% from Station 3134+00 to 3137+00

The standard ditch will provide an adequate depth from 3137+00 to the top of the hill. Checking the cross section plots shows that the earthwork to construct the standard ditch will not extend beyond the proposed R/W line.

Step 4 – Compute the Flow Depths and Velocities. These values were calculated in the description of the previous step. In most cases, the designer will be iterating through steps 3 and 4 as the ditch is designed.

The ditch checks that should be included in the Drainage Documentation to prove the design are shown in Figure 3-6.

HYDRAULIC WORKSHEET FOR ROADSIDE DITCHES — Sheet 1 of 1

Road: New Road — Prepared by: XXX — Date: 4/1/09

Project Number: 1234567 — Checked by: YYY — Date: 4/1/09

STATION TO STATION	SIDE	% Slope	Drain Area	"C"	Tc	I_{10}	Q (cfs)	Ditch Section F.S.	Ditch Section B.W.	Ditch Section B.S.	"n"	"d"	Vel (fps)	Ditch Lining	Side Drain Pipe Dia	Remarks
3126+50	LT	0.20	2.61	0.75	15	6.5	12.7	6	5.0	4	0.042	1.03	1.2	SOD		TW El. will control
3127+00	LT	0.10					12.7	6	5.0	4	0.042	1.22	0.9	Seed & Mulch		TW El. will control
3134+00	LT	0.10	1.79	0.75	10	7.6	10.2	6	5.0	4	0.042	1.10	0.9	Seed & Mulch		
3134+00	LT	2.39					10.2	6	3.0	4	0.6	0.59	2.2	SOD		

Note: F.S. = Front Slope B.W. = Bottom Width B.S. = Back Slope

Figure 3-6
Roadside Ditch Design Example

3.8 Side Drains

Continuous flow in a roadside ditch can be interrupted by side street/road connections and/or driveway connections to the project roadway. Even a limited access roadway, such as an Interstate highway, may have an occasional access driveway that will impede roadside ditch flow, especially at or near adjacent stormwater pond locations. Ditch flow continuity is maintained through such obstructions via roadside ditch culverts or side drains.

A side drain is then a class of culvert pipe that can transport flow through fill placed in a roadside ditch. A side drain is normally aligned parallel or nearly parallel to the project roadway and along the flowline of the ditch. However, if a side drain is located under a public road connecting to the project roadway, the culvert should be identified and hydraulically sized as a cross drain (see the Culvert Design Handbook). Side drains and cross drains are similar in many ways, but there are some differences in design analysis requirements, materials, and end treatment. Cross drains have to meet more rigorous criteria for some parameters.

3.8.1 Design Analysis Requirements for Side Drains

A side drain is sized for the storm frequency required to design the roadside ditch that contains the side drain (usually the 10 year frequency, as mentioned in Section 3.6.1). The side drain design flow is determined from application of the same hydrologic method used to compute the corresponding ditch design flows (usually the Rational Equation described in Section 2.2.3 of the Hydrology Handbook). The side drain pipe dimensions are then determined via the inlet-control/outlet-control procedure described in Chapter 6 of the Culvert Design Handbook. (Note: The FHWA HY-8 computer software is one of several computer programs capable of applying this procedure to the side drain design data.)

The design flow for a side drain is normally developed in the design calculations spreadsheet or worksheet for the roadside ditch that contains the side drain. (Figure 2-1 of the Drainage Manual depicts such a ditch design worksheet.) The design flow and surface water depth for the ditch section at the upstream end of the side drain are determined in the ditch calculations, and this ditch flow is the side drain design inflow as well. This flow is also typically the design flow for the ditch section at the downstream end of the side drain, and must be accounted for in the calculations for the remainder of the downstream ditch length. Of course, if additional flow enters the side drain between its upstream and downstream ends, this additional flow must also be appropriately accounted for in both the side drain hydraulic design and in the downstream ditch design calculations.

The tailwater elevation at the culvert outlet needs to be determined. Since the culvert is usually placed through fill in the roadside ditch, the ditch calculations downstream of the culvert are used to determine the tailwater. The culvert tailwater will be the normal depth in the downstream ditch unless the tailwater for the ditch controls the water surface elevation at the side drain outlet. Refer to Section 3.7 for more discussion on tailwater.

The hydraulic calculations for a side drain can then be generated, using the procedure described above, to determine the pipe dimensions needed to safely pass the design flow to the downstream ditch segment. These Side Drain Calculations should be included in the Drainage Documentation Report as either a separate section or as part of the Ditch Calculations section.

Note that the surface water depth computed for culvert flow at the upstream end of a side drain will generally be larger than the depth computed for ditch flow at that location. If the difference in this flow depth is not significant, the ditch flow depths upstream from the side drain should be evaluated and adjusted (if appropriate) for the “flat pool” that will be established in the ditch by the higher of the two water surface elevations. If the difference in surface water flow depth at the side drain is substantial and the ditch design is sensitive to actual flow depths, a backwater analysis may be needed rather than the “flat pool” approximation in determining the actual flow depth estimates.

3.8.2 Material Requirements

In general, side drains are not considered to be as critical as cross drains. Therefore, material service life requirements for side drains are less stringent than for cross drains. Consult Chapter 6 of the FDOT Drainage Manual, the FDOT Standard Specifications for Road and Bridge Construction, the Optional Pipe Materials Handbook, and the appropriate District Drainage Engineer for any clarification needed on pipe materials acceptable for use as side drains. Culvert and ditch calculations may show the need for two allowable pipe sizes, depending on the Manning’s roughness coefficients of the optional pipe materials for the side drain.

3.8.3 End Treatment

The only allowable side drain end treatment is the mitered end section (Index No. 273). Due to the normal side drain alignment and close proximity to the project roadway (usually within the clear zone), Standard Index 273 specifies that grates be installed for the larger pipe sizes. The grates are intended to provide a measure of safety for errant vehicles that encounter the end treatment. The grates, however, will also potentially collect debris and will increase the entrance loss coefficient, Ke, from 0.7 to 1.0 for the mitered end section. When a grate is likely to be used, the following items should be considered:

- The design engineer should recognize that the specification of a grate could increase the required side drain size (due to the increase in Ke).
- In critical hydraulic locations, grates shall not be used until potential debris transport has been evaluated by the design engineer and appropriate adjustments made. Vegetated ditch grades in excess of 3 percent, or pipe with

less than 1.5 feet of cover, or paved ditch grades in excess of 1 percent will require such an evaluation.

- The design engineer shall determine highly corrosive locations and specify in the plans when the grates shall be hot-dipped galvanized after fabrication.

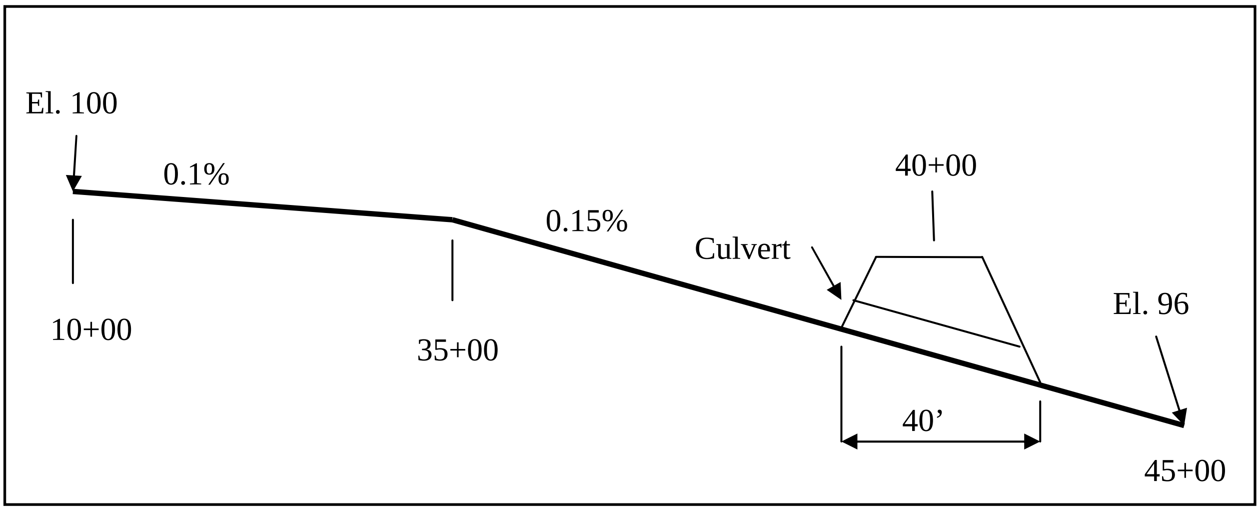
Example 3.2 – Side Drain Design

Problem Statement:
A driveway is included in the design of the left roadside ditch for a new two lane rural roadway segment. Figure 3-7 depicts the typical section for the left side of the roadway. The ditch extends and flows from Station 10+00 to Station 45+00, with the centerline of the driveway located at Station 40+00. The width of the proposed driveway base at the ditch flowline is 40', and the ditch section is uniform throughout its length with a 2' allowable depth below the left top-of-bank. At its upstream and downstream ends, the ditch flowlines must match elev. 100.0' and elev. 96.0', respectively. The ditch longitudinal slopes are 0.1% from Station 10+00 to Station 35+00, and 0.15% from Station 35+00 to Station 45+00. The site is located in FDOT IDF-curve Zone 7, and the natural ground slopes away from the left top-of-bank of the ditch section.

Determine the required side drain diameter.

Design Approach:
First, develop the ditch design calculations to determine the side drain design inflow at Station 39+80. These calculations are shown on Figure 3-6, and indentify a side drain design flow of 4.60 cfs.

Next, refer to Chapter 6 of the Culvert Design Handbook for the side drain hydraulic design procedure. Use either the inlet control and outlet control nomographs from FHWA HDS-5, or software such as HY-8 to develop the required side drain size.



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HYDRAULIC WORKSHEET FOR ROADSIDE DITCHES — Sheet 1 of 1

Road: New Road — Prepared by: XXX — Date: 4/1/09

Project Number: 1234567 — Checked by: YYY — Date: 4/1/09

STATION TO STATION	SIDE	% Slope	Drain Area	"C"	Tc	I_{10}	Q (cfs)	Ditch Section F.S.	Ditch Section B.W.	Ditch Section B.S.	"n"	"d"	Vel (fps)	Ditch Lining	Side Drain Pipe Dia	Remarks
10+00 - 35+00	LT	0.10	2.75	0.47	60.1	3.24	4.19	6	3.0	4	0.042	0.82	0.72	Seed & Mulch		
35+00 - 39+80	LT	0.15	3.31	0.47	69.5	2.96	4.60	6	3.0	4	0.042	0.78	0.85	Seed & Mulch		Drain Area includes 1/2 of driveway width
39+80 - 40+20	LT						4.60								18"	See Side Drain Calcs for details
40+20 - 45+00	LT	0.15	3.86	0.47	78.6	2.72	4.93	6	3.0	4	0.042	0.81	0.87	Seed & Mulch		Drain Area includes 1/2 of driveway width

Note: F.S. = Front Slope B.W. = Bottom Width B.S. = Back Slope

Figure 3-7
Side Drain Design Example

Chapter 4

Channel Linings

As stated in Section 2.4.3 of the Drainage Manual, the design of open channels shall consider the need for channel linings. In addition, most maintenance problems in channels are related to erosion and sloughing. Channel linings will often be the solution to those problems. Standard lining types can be found in the Standard Indexes as well as the Standard Specifications. The two main classifications of open channel linings are flexible and rigid. Flexible linings include vegetative linings such as grass, rubble riprap and geotextile or interlocking concrete grids. Rigid linings include concrete, asphalt and soil-cement. From an erosion control standpoint, the primary difference between rigid and flexible channel linings is their response to changes in channel shape (i.e. the width, depth and alignment). For most artificial channels, the most desirable lining is natural, emerging vegetation, with grass used to provide initial and long-term erosion resistance.

The following are examples of lining materials in each classification.

1. Flexible linings
   a. Grasses or natural vegetation
   b. Rubble riprap
   c. Wire-enclosed riprap (gabions)
   d. Turf reinforcement (non-biodegradable)
2. Rigid Linings:
   a. Cast-in-place concrete or asphaltic concrete
   b. Soil cement and roller compacted concrete
   c. Grout filled mattresses
   d. Partially grouted riprap
   e. Articulated concrete blocks

4.1 Flexible Linings

Flexible linings have several advantages compared to rigid linings. They are generally less expensive, permit infiltration and exfiltration and can be vegetated to have a natural appearance. Flow in channels with flexible linings is similar to that found in natural small channels. Natural conditions offer better habitat opportunities for local flora and fauna. In many cases, flexible linings are designed to provide only transitional protection against erosion while vegetation establishes and becomes the permanent lining of the channel; flexible channel linings are best suited to conditions of moderate shear stresses. Channel reaches with accelerating or decelerating flow (expansions, contractions, drops and backwater) and waves (transitions, flows near critical depth, and shorelines) will require special analysis and may not be suitable for flexible channel linings.

4.1.1 Vegetation

Vegetative linings consist of seeded or sodded grasses placed in and along the channel as well as naturally occurring vegetation. Vegetation is one of the most common and most desirable channel linings for an artificial channel. It stabilizes the body of the channel, consolidates the soil mass of the bed, checks erosion on the channel surface, and controls the movement of soil particles along the channel bottom. Vegetative channel lining is also recognized as a best management practice for stormwater quality design in highway drainage systems. The slower flow of a vegetated channel helps the uptake of highway runoff contaminants (particularly suspended sediments) before they leave the highway right of way and enter streams.

There are conditions which vegetation may not be acceptable and other linings should be considered. These conditions include but are not limited to:

- Standing or continuous flowing water
- Areas which do not receive the regular maintenance necessary to prevent domination by taller vegetation
- Lack of nutrients and excessive soil drainage
- Areas where sod will be excessively shaded

The Department operates on the premise that with proper seeding and mulching during construction, maintenance of most ditches on normal sections and grades can be handled economically until a growth of grass becomes established. The use of temporary erosion control measures in ditches with low velocities will provide time for grassing and mulching to establish a vegetative ditch. When velocities exceed those for bare soils, seeding and mulching should not be used.

Sodding is recommended when the design velocity exceeds the value permitted for the bare base soil conditions, but is less than 4 feet per second. Lapped or shingle sod is recommended when the design velocity exceeds that for sod (4 feet per second), and is suitable with velocities up to 5.5 feet per second.

4.1.2 Other Flexible Linings

Flexible linings are usually less expensive than rigid linings, provide a safer roadside, and have self-healing qualities which reduce maintenance. They also allow the infiltration and exfiltration of water.

Rubble Riprap

After grass, rubble riprap is the most common type of flexible lining. It presents a rough surface which can dissipate energy and mitigate velocity increases. There are two standard types of rubble riprap. Ditch lining rubble riprap should be used in standard or typical ditches or channels. It consists of smaller stone sizes which reduces construction costs over bank and shore rubble. Bank and shore rubble riprap should be limited to uses such as revetments and linings along stream banks and shorelines where extreme flows or wave action occurs.

Limited right-of-way and unavailability of material may restrict the use of this type of flexible lining. Rubble riprap is placed on a filter blanket and prepared slope to form a well-graded mass with a minimum of voids. Riprap and gabion linings can perform in the initial range of hydraulic conditions where rigid linings are used. Stones used for riprap and gabion installations preferably have an angular shape that allows stones to interlock. These linings usually require a filter material between the stone and the underlying soil to prevent soil washout to prevent migration of fines. A bedding stone layer may also be needed to protect the filter fabric.

Gabion Mats

Wire-enclosed riprap (gabions) is a wire container or enclosure structure that binds units of the riprap lining together. The wire enclosure normally consists of a rectangular container made of steel wire woven in a uniform pattern, and reinforced on corners and edges with heavier wire. The containers are filled with stone, connected together, and anchored to the channel side slope. The forms of wire-enclosed riprap vary from thin mattresses to boxlike gabions. Wire-enclosed riprap is typically used when rubble riprap is either not available or not large enough to be stable. Although flexible, gabion movement is restricted by the wire mesh. The wire mesh must provide an adequate service life. If the wire mesh fails, the individual stones will migrate.

Turf Reinforcement

Depending on the application, materials, and method of installation, turf reinforcement may serve a transitional or long-term function. The concept of turf reinforcement is to provide a structure to the soil/vegetation matrix that will both assist in the establishment of vegetation and provide support to mature vegetation. Two types of turf reinforcement are commonly available: soil/gravel methods and turf reinforcement mats (TRMs).

Soil/gravel turf reinforcement is to mix gravel mulch into on-site soils and to seed the soil-gravel layer. The rock products industry provides a variety of uniformly graded gravels for use as mulch and soil stabilization. A gravel/soil mixture provides a non-degradable lining that is created as part of the soil preparation and is followed by seeding.

A TRM is a non-degradable rolled erosion control product (RECP) composed of UV stabilized synthetic fibers, filaments, netting and/or wire mesh processed into a three-dimensional matrix. TRMs provide sufficient thickness, strength and void space to permit soil filling and establishment of grass roots within the matrix. One limitation to the use of TRMs is in the areas where siltation is a problem. When the ditch is cleaned by maintenance there is likelihood that the geofabric will be snagged and pulled out by the equipment.

4.2 Rigid Linings

Rigid linings are generally constructed of concrete, asphalt, or soil-cement pavement whose smoothness offers a higher capacity for a given cross-sectional area. Higher velocities, however, create the potential for scour at channel lining transitions from the rigid lining back to the grass lining. A rigid lining can be destroyed by flow undercutting the lining, channel headcutting, or the buildup of hydrostatic pressure behind the rigid surfaces. When properly designed, rigid linings may be appropriate where the channel width is restricted. Rigid linings are useful in flow zones where high shear stress or rapidly varied or turbulent flow conditions exist, such as at transitions in channel shape or at an energy dissipation structure.

Rigid linings are particularly vulnerable to a seasonal rise in water table that can cause a static uplift pressure on the lining. If a rigid lining is needed in such conditions, a reliable system of under drains and weep holes should be a part of the channel design. The migration of soil fines into filter layers should be evaluated to ensure that the ground water is discharged without filter clogging or collapse of the underlying soil. A related case is the build up of soil pore pressure behind the lining when the flow depth in the channel drops quickly. Use of watertight joints and backflow preventers on weep holes can help to reduce the build up of water behind the lining.

Section 2.4.4 of the Drainage Manual states that when concrete linings are to be used where soils may become saturated, the potential for buoyancy shall be considered due to the uplift water pressure. The total upward force is equal to the weight of the water displaced by the channel. The uplift pressure is resisted by the total weight of the lining. When the weight of the lining is less than the uplift pressure, the channel is unstable.

Acceptable countermeasures include:

- Increasing the thickness of the lining to add additional weight.
- For sub-critical flow conditions, specifying weep holes at appropriate intervals in the channel bottom to relieve the upward pressure on the channel.
- For super-critical flow conditions, using subdrains in lieu of weep holes.

4.2.1 Cast-in-Place Concrete and Soil Cement Concrete

The minimum standard paved section is a 4-foot bottom width ditch. A 3-foot bottom width paved section can be used if the channel is beyond the anticipated path of errant vehicles and hydraulic capacity is adequate. Asphalt linings have limited use since they are often damaged or destroyed by routine maintenance activities. Filter fabric is required to prevent soil loss through pavement cracks.

Despite the non-erodible nature of concrete linings, they are susceptible to failure from foundation instability. The major cause of failure is undermining that can occur in a number of ways. Inadequate erosion protection at the outfall, at the channel edges, and on bends can initiate undermining by allowing water to carry away the foundation material and leaving the channel to break apart. Concrete linings may also break up and deteriorate due to conditions such as a high water table or swelling soils that exert an uplift pressure on the lining. Once a rigid lining is locally broken and displaced upward, the lining continues to move due to dynamic uplift and drag forces. The broken lining typically forms large, flat slabs that are particularly susceptible to these forces.

4.2.2 Grout Filled Mattresses

Grout filled mattresses are the result of pumping a concrete mix into fabric envelopes or cases. The advantages of using grout filled mattresses are that they reduce construction time by eliminating the need for wooden forms and expensive lifting machines and also allow the concrete to be pumped and cured below the water line. There are two commonly used types of grout filled mattresses; articulating block and filter point.

4.2.2.1 Articulating Block

Articulated block grout filled mattresses consist of rectangular concrete blocks that are cast in place. The block pattern provides a higher coefficient of hydraulic friction for the use of energy dissipation. The blocks may be linked together with a reinforcing cable if required. The cables allow the lining to articulate with a change in soil or water conditions. This is particularly important for slopes that are subject to severe underscour or consolidation. Non-reinforced articulated block linings should only be used when soil conditions predict minimal settlement.

4.2.2.2 Filter Point

Filter point grout filled mattresses consist of a dual wall fabric that is injected with concrete. This type of grout filled mattress is characterized by a deeply cobbled surface. The filter points woven into the fabric provide a means for groundwater to escape and to provide release for the hydrostatic pressure. Filter Point fabrics provide a higher coefficient of friction to promote energy dissipation.

4.3 Velocity and Shear Stress Limitations

HEC-15 provides a detailed presentation of stable channel design concepts of roadside and median channels. This section provides a brief summary of significant concepts.

Stable channel design concepts provide a means of evaluating and defining channel configurations that will perform within acceptable limits of stability. For most highway drainage channels, bank instability and lateral migration cannot be tolerated. Stability is achieved when the material forming the channel boundary effectively resists the erosive forces of the flow. Principles of rigid boundary hydraulics can be applied to evaluate this type of system.

Both velocity and tractive force methods have been applied to the determination of channel stability. Permissible velocity procedures are empirical in nature, and have been used to design numerous channels in Florida and throughout the world. However, tractive force methods consider actual physical processes occurring at the channel boundary and represent a more realistic model of the detachment and erosion processes.

The hydrodynamic force created by water flowing in a channel causes a shear stress on the channel bottom. The bed material, in turn, resists this shear stress by developing a tractive force. Tractive force theory states that the flow-induced shear stress should not produce a force greater than the tractive resisting force of the bed material. This tractive resisting force of the bed material creates the permissible or critical shear stress of the bed material. In a uniform flow, the shear stress is equal to the effective component of the gravitational force acting on the body of water parallel to the channel bottom. The average shear stress is equal to:

$$\tau = \gamma R S \qquad (4\text{-}1)$$

where,
τ = average shear stress, lb/ft^2
γ = unit weight of water,62.4 lb/ft^3
R = hydraulic radius, ft
S = average bed slope or energy slope,ft/ft

The maximum shear stress for a straight channel occurs on the channel bed and is less than or equal to the shear stress at maximum depth. The maximum shear stress is computed as follows:

$$\tau_d = \gamma d S \qquad (4\text{-}2)$$

where,
τ_d = maximum shear stress, lb/ft^2
d = maximum depth of flow, ft
S = channel bottom slope, ft/ft

Velocity limitations for artificial open channels should be consistent with stability requirements for the selected channel lining. As indicated above, seed and mulch should only be used when the design velocity does not exceed the allowable velocity for bare soil. Maximum shear stress values and allowable velocities for different soils are presented in Table 2.3 of the Drainage Manual. When design velocities exceed those acceptable for bare soil, sod, or lapped sod, flexible or rigid linings should be considered. Maximum velocities for these lining types are summarized in Table 2.4 of the Drainage Manual.

Side Slope Stability
The shear stress on the channel sides is generally less than the maximum shear stress calculated on the channel bottom but needs to be considered when determining the height of a channel lining along the side slope of the channel. The maximum shear stress on the side of a channel is given by:

$$\tau_s = K_1 \tau_d \qquad (4\text{-}3)$$

where,
τ_s = side shear stress on the channel, lb/ft^2
K_1 = ratio of channel side to bottom shear stress
τ_d = shear stress in channel at maximum depth lb/ft^2

The value K_1 depends on the size and shape of the channel. For parabolic channels the shear stress at any point on the side slope is related to the depth at that point and can be calculated using Equation 4.2. For trapezoidal and triangular channels, K_1 is based on the horizontal dimension 1: Z (V: H) of the side slopes.

$K_1 = 0.77$	$Z \le 1.5$
$K_1 = 0.066Z + 0.67$	$1.5 < Z < 5$
$K1 = 1.0$	$5 \le Z$

Use of side slopes steeper than 1:3 is not encouraged for flexible linings other than rip rap or gabions because of the potential for erosion at the side slopes. Steep side slopes are allowable within a channel if cohesive soil conditions exist.

Maintenance considerations

Maintenance of the channel will also need to be considered when choosing a channel lining. The channel will need to be accessible by mowers and trucks.

Mowing

Side slopes of vegetated channels will need to be traversable for mowing equipment and crews. The maximum traversable slope for this equipment is 1:4.

Access across channel

If the channel is lined with riprap rubble and there is a vegetated buffer on the backside of the channel along the right of way. Access across the channel may not be possible due to the irregularity of the riprap and maintenance of the vegetation may become unpractical.

4.4 Application Guidance for Some Common Channel Linings

4.4.1 Rubble Riprap

Types

- **Ditch Lining** – Flexible layer or facing of rock placed on a filter blanket and prepared slope used to line a ditch or channel for protection from erosion.

- **Bank and Shore** – Flexible layer or facing of rock placed on a bank or shore and prevent erosion or scour of the embankment or a structure.

What is its purpose?

Rip rap rubble is to be used in channels, along embankments, or around structures which are vulnerable to erosion or scour.

Where and how is it commonly used?

- **Ditch Lining** – This is used to line ditches and channels to protect slopes from erosion.

- **Bank and Shore** – This is used as a flexible revetment to line banks and shores subject to erosion.

When should it be installed?

- **Ditch Lining** – This should be installed in channels with moderate shear stresses. To prevent uplifting forces on the lining, the filter requires adequate permeability.

- **Bank and Shore** – This should be used to protect banks or shores with flows which are generally greater than 50 ft^3/s or subject to wave action.

When should it not be installed?

- **Bank and Shore** – This should not be installed when ditch lining methods are applicable.

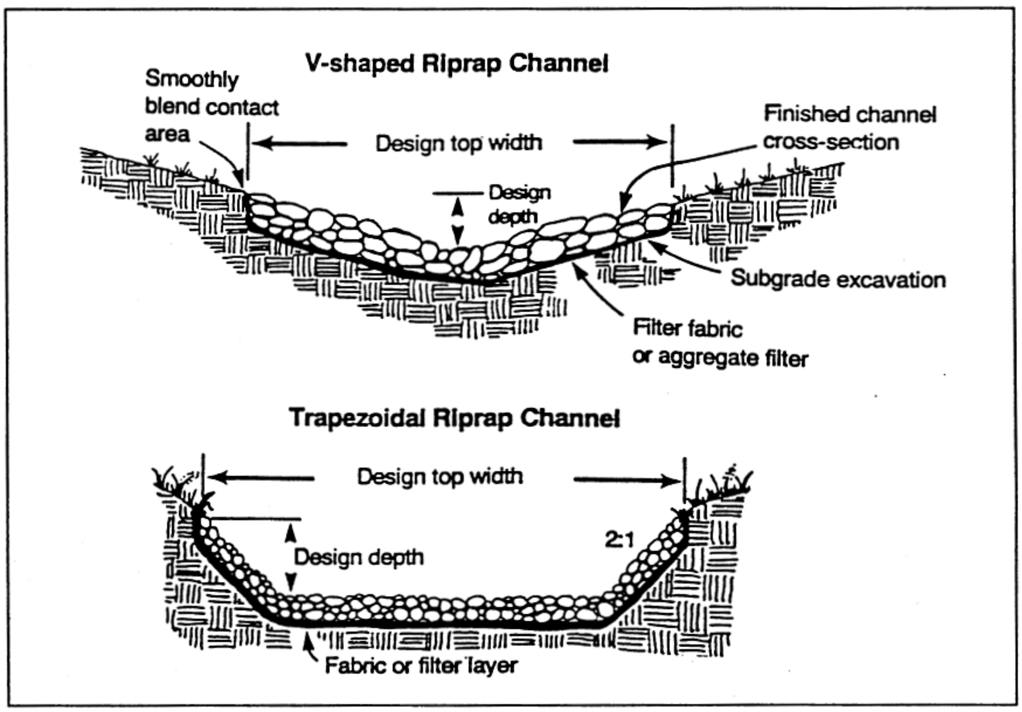
Advantages and Disadvantages

ADVANTAGES

- Flexible
- Not weakened by minor shifting caused by settlement
- Easily repaired by additional rock placement
- Simple construction method
- Recoverable/reusable
- Long-term or temporary installations

DISADVANTAGES

- Hauling and installation costs
- Prohibits maintenance equipment from traversing channels
- If hand placement is required then labor is intensive
- Vegetation growth can hinder inspections



(Source: http://www.dlr.enr.state.nc.us/pages/publications.html#eslinks)
North Carolina Department of Environmental and Natural Resources

Figure 4-1. Rip rap lined channel cross sections

4.4.2 Grout Filled Mattresses

Types

- **Filter Point Linings** – Grout filled mattresses for concrete with filtering points that provide for the relief of hydrostatic pressures.

- **Articulating Block Mats** – Grout filled mattresses for concrete that consist of a series of compartments that are connected by high strength revetment cables to facilitate articulation.

What is its purpose?

Grout filled mattresses, filter point or articulating, are to be used for slopes or areas that are subject to severe to moderate erosion problems.

Where and how it is commonly used?

- **Filter Point Linings** – These are used in ditches, channels, canals, streams, rivers, ponds, lakes, reservoirs, marinas, and ports/harbors to reduce the impact of erosion.

- **Articulating Block Mats** – These are used to protect shorelines, rivers, canals, lakes, reservoirs, underwater pipelines, bridge piers and other marine structures to reduce the impact of erosion.

When should it be installed?

- **Filter Point Linings** – These should be installed where there are moderate to severe erosion problems and are subjected to hydrostatic uplift pressures. Also where there is a need to allow water to permeate into the soil and not remain wet.

- **Articulating Block Mats** – These should be installed where there are moderate to severe erosion problems due to the wave action, propeller wash, ship wakes, and high velocity flows.

When should it not be installed?

- **Filter Point Linings** – These should not be used in ditches or channels that are subject to changes in soil conditions such as erosion under the mat or consolidation.

- **Articulating Block Mats** – These should not be used in ditches or channels that are subject to hydrostatic pressure due to changes in groundwater conditions.

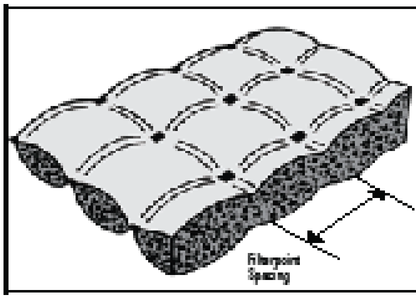
Advantages and Disadvantages

ADVANTAGES

- Adapts easily to contours
- Easy to install
- Permeable
- Reduced uplift pressure
- Can be installed under the water line

DISADVANTAGES

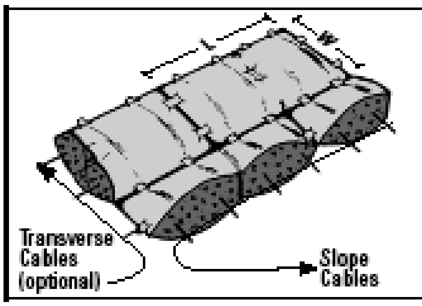
- Needs to be installed on a prepared slope
- Not aesthetically pleasing
- Easily undermined if not toed properly



Designation Style	CAST-IN-PLACE							
	Filterpoint Spacing		Average Thickness*		Coverage Per		Dry Weight**	
	in.	mm	in.	mm	Y^3 Mortar	M^3 Mortar	lb / ft^2	kg / m^2
5" FPNN	5	127	2.2	56	135 ft^2	16.39 m^2	25	122
8" FPNN	8	200	4	100	75 ft^2	9.11 m^2	45	220
10" FPNN	10	250	6	150	50 ft^2	6.07 m^2	68	330

(Source: http://www.fabriform1.com)
Construction Techniques, Inc.

Figure 4-3. Filter Point Linings



Designation Style	CAST-IN-PLACE							
	Block Size (LxW)*		Average Thickness**		Coverage Per		Dry Weight***	
	in.	mm	in.	mm	Y^3 Mortar	M^3 Mortar	lb / ft^2	kg / m^2
4" ABNN	20 x 12	500 x 300	4	100	75 ft^2	9.11 m^2	45	220
6" ABNN	20 x 20	500 x 500	6	150	50 ft^2	6.07 m^2	68	330
8" ABNN	40 x 20	1000 x 500	8	200	38 ft^2	4.55 m^2	90	440

(Source: http://www.fabriform1.com)
Construction Techniques, Inc.

Figure 4-4. Articulating Block Mats

4.4.3 Gabions

Types

- **Gabion Mats** – Wire mesh mats filled with stones.

- **Gabion Baskets** – Wire mesh baskets filled with stones.

What is its purpose?

Rock filled baskets or mattresses that are used to line large ditches, channels, canals and coastal shores for stabilization and protection.

Where and how it is commonly used?

Gabion Mats – These are used in ditches, channels, canals, streams, rivers, ponds, lakes, reservoirs, marinas, and ports/harbors to reduce the impact of erosion.

When should it be installed?

Gabion Mats – These should be installed in large areas where there are moderate to severe erosion problems due to extreme velocities. Also where there is a need to allow water to permeate into the soil and not remain wet.

When should it not be installed?

- **Gabion Baskets** – Small areas subject to low velocities and when a temporary situation exists.

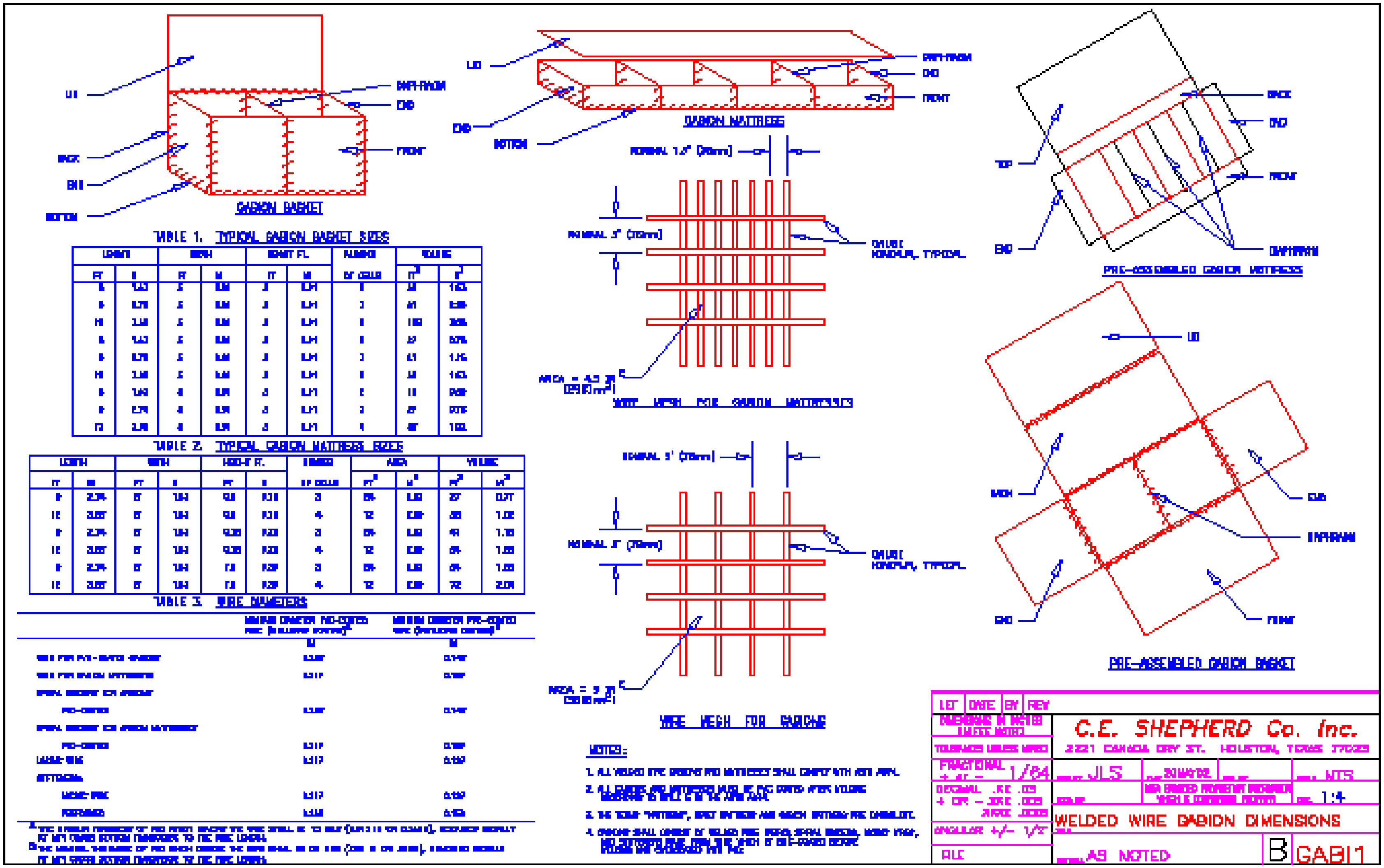
Advantages and Disadvantages

ADVANTAGES

- Protect seed mix from eroding when used.
- Permeable.
- Increase retention of soil moisture.
- Permit the growth of vegetation
- Able to span minor pockets of bank subsidence without failure

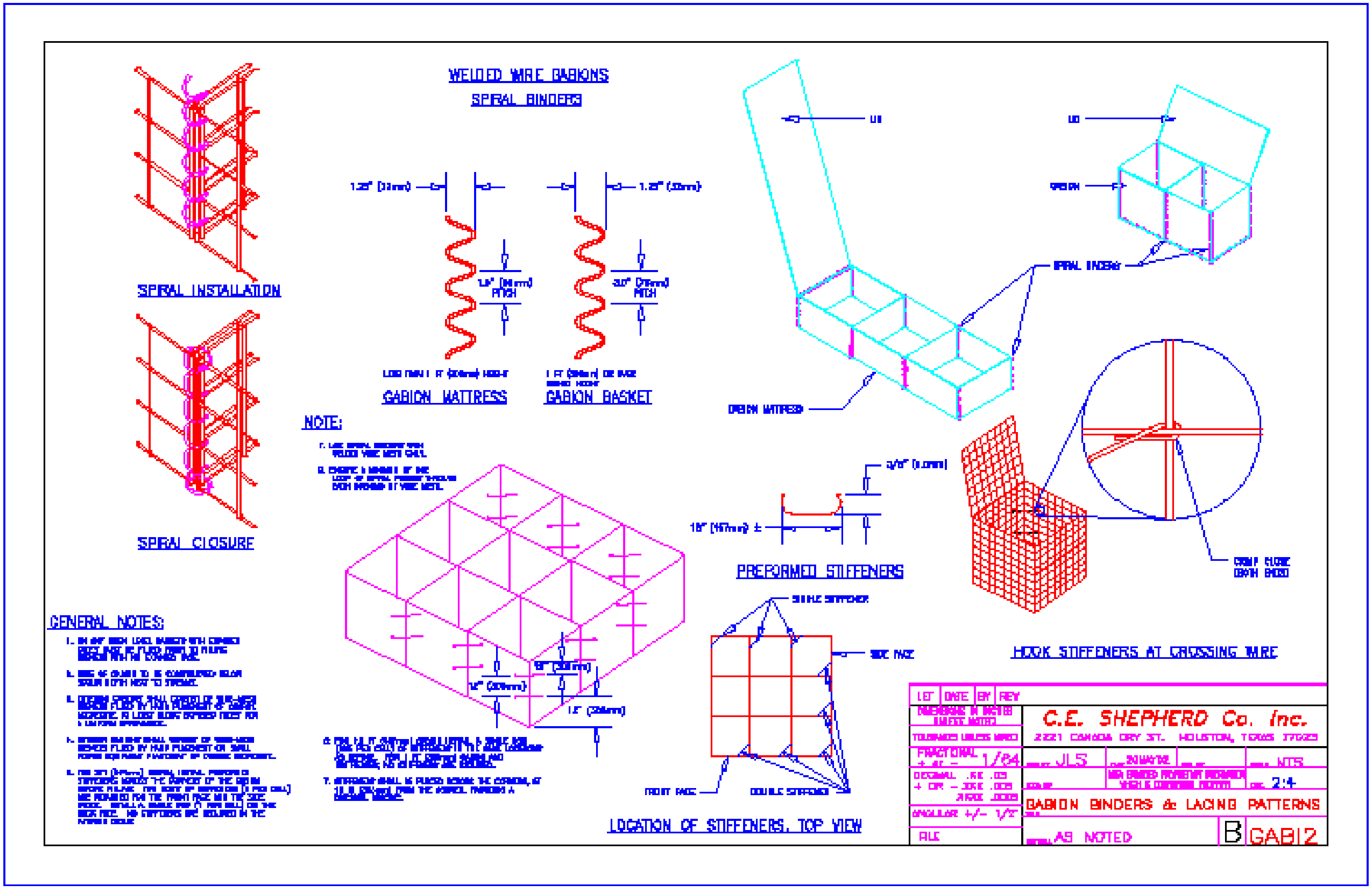
DISADVANTAGES

- Cost of installation
- Susceptibility of the wire baskets to corrosion and abrasion damage
- More difficult and expensive to repair
- Less flexible than standard rip rap



(Source: http://www.gabions.net/downloads.html)
Modular Gabion Systems, a division of C.E. Shepherd Company

Figure 4-5. Gabion Dimensions



(Source: http://www.gabions.net/downloads.html)
Modular Gabion Systems, a division of C.E. Shepherd Company

Figure 4-6. Gabion Binding

4.4.4 Soil Stabilizers

Types

- **Turf Reinforcement Mats** – A long term non-degradable mat composed of UV stabilized, non-degradable, synthetic fibers, nettings and/or filaments.

- **Erosion Control Blankets** – A temporary degradable mat composed of processed natural or polymer fibers mechanically, structurally or chemically bound together to form a continuous matrix.

What is its purpose?

To protect disturbed slopes and channels from wind and water erosion. The blanket materials are natural materials such as straw, wood excelsior, coconut, or are geotextile synthetic woven materials such as polypropylene.

Where and how it is commonly used?

- **Turf Reinforcement Mats** – These are used on ditch slopes and fill slopes to reduce the impact of erosion for long periods of construction.

- **Erosion Control Blankets** – These are used on ditch slopes and fill slopes to reduce the impact of erosion during short periods of construction.

When should it be installed?

- **Turf Reinforcement Mats** – Where there are low velocities of flow.

- **Erosion Control Blankets** – Where there are low velocities of flow and where there are sensitive environmental areas.

When should it not be installed?

- **Turf Reinforcement Mats** – Not to be installed for permanent situations and where there are high velocities of flow.

- **Erosion Control Blankets** – Not to be installed for permanent situations and where there are high velocities of flow.

Advantages and Disadvantages

ADVANTAGES

- Adapts easily to contours.
- Easy to install.
- Permeable.
- Reduced uplift pressure.

DISADVANTAGES

- Cost
- Maintenance equipment can damage or pull out

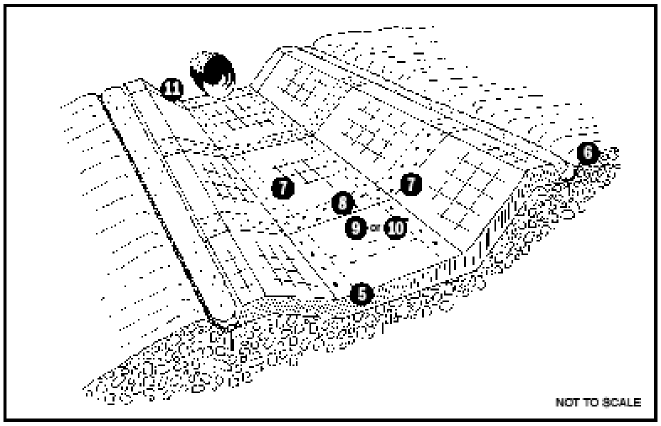


Figure 4-7. Erosion Control Mat in Channel
(Source: http://www.geotextile.com)
Propex Geosynthetics

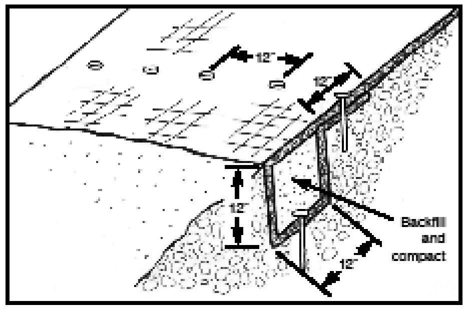
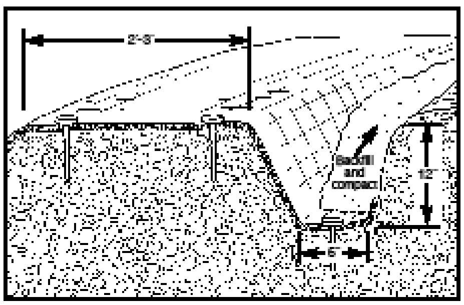


Figure 4-8. Initial Anchor (Downstream)
(Source: http://www.geotextile.com)
Propex Geosynthetics



(Source: http://www.geotextile.com)
Propex Geosynthetics

Figure 4-9. Longitudinal Anchor Trench Detail (Trapezoidal Channel)

Chapter 5

Drainage Connection Permitting and Maintenance Concerns

5.1 Drainage Connection Permitting

Adjacent property owners must obtain a Drainage Connection Permit from FDOT according to Section 334.044(15), F.S., Chapter 14-86, F.A.C., Rules of the Department of Transportation, when developing their property. In general terms, the Drainage Connection Permit ensures that the development will not overload the Departments stormwater conveyance systems and cause flooding on either the roadway or other downstream properties. For more information on Drainage Connection Permits, refer to the Drainage Connection Permitting Handbook. This section will discuss several aspects of the Department's ditches that should be considered during the Drainage Connection Permitting process.

5.1.1 Roadside Ditch Impacts

Discharges to the roadside ditch from the proposed development will be limited by the Permit so that the ditch flow will not be increased. However, the proposed development can physically impact the roadside ditch by placing or widening turnouts to the property or by widening the roadway to add turn lanes.

If the roadside ditch is a linear treatment pond, then any reduction in the volume of the ditch could violate the conditions of the permit obtained for the facility. The simplest way to resolve this issue is to rework the ditch so that any volume lost as a result of the development is replaced. This may require that the property owner donate some property to the Department to provide an area to rework the ditch.

Even if the roadside ditch is not a linear treatment facility, the capacity of the ditch must be maintained. A side drain will be needed to convey the ditch flow from one side of the turnout to the other, unless the turnout is located at a high point in the ditch and the flow is away from the turnout in both directions. An added turn lane may require that the roadside ditch be relocated. The relocated portion of the ditch should have the same capacity or more than the existing ditch. If the existing R/W is not wide enough to accommodate the relocated ditch, then R/W may need to be donated to FDOT for the ditch. A turnout requiring a side drain and a turn lane requiring donated R/W for the ditch relocation are shown in Figure 5-1.

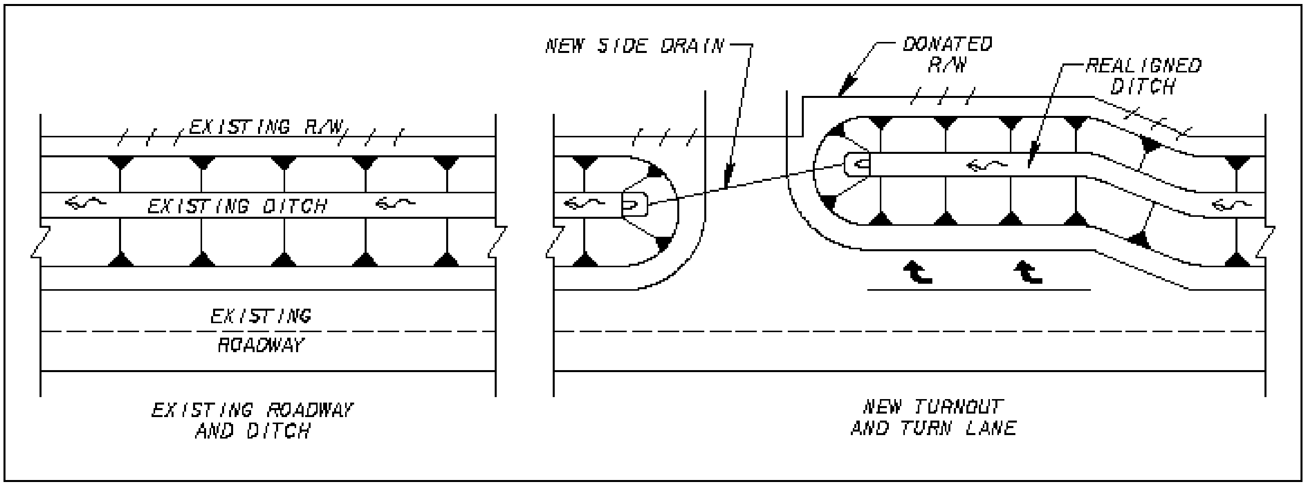


Figure 5-1 Effect of Adjacent Development on a Roadside Ditch

In some cases, the developer may need to add a left turn lane. Widening the road to accommodate the left turn lane may also affect the ditch on the opposite side of the road from the development. Often, the developer will not own the property on both sides of the road. In this case, the roadside ditches and roadway must be redesigned to accommodate the new turn lanes in such a way as to require donated R/W on the new development's side of the road.

The flow lines of the side drain should match the existing ditch. The reviewer should also ensure that the flow lines of the new side drain are higher than the next side drain downstream and lower than the next side drain upstream to avoid temporary ponding in the ditch.

The side drain should also be sized properly. Some judgments about the size of the pipe can be made by looking at the side drains up and downstream of the new drive. However, the side drain should be analyzed to ensure the new pipe does not cause the water levels to pop out of the ditch. In some cases the design discharge for the ditch can be obtained from the old plans for the roadway. Or the flow can be calculated by determining the drainage area and performing the proper hydrologic calculations, typically the rational equation. More details on these calculations can be found in the Hydrology Handbook. The losses through the pipe can be calculated using methods given in the Cross Drain Handbook. Additional sizing considerations are discussed in Section 3.8.

Another consideration when adding new side drains is the proximity of other existing sidedrains. If side drains are too close to each other then the hydraulic losses can be too large. The general requirement is that the end sections of two side drains in series should be at least 25' apart. If the distance is less than 25', then the area should be enclosed and an inlet added to collect the runoff from the area between the turnouts.

Potential erosion at the infall point of the connection should be evaluated, especially for pipe connections. Calculating the outlet velocity from a pipe is explained in the Cross Drain Handbook and outlet erosion protection criteria can be found in the Drainage Manual. Refer to Chapter 4 for channel linings.

5.1.2 Median Ditch Impacts

The median ditch can be impacted by a development if the Department allows a new median opening or left turn lane.

Unless a new median opening is placed at the high point in the median ditch, or close enough to the high point that the ditch can be regraded to flow away from the new median opening in both directions, then the flow in the ditch will be blocked by the new opening. Figure 5-2 shows a typical situation where there is an existing median opening at the high point in the median ditch and the ditch flows to a median drain which consists of a Ditch Bottom Inlet, pipe, and Endwall. The median drain discharges runoff from the median to keep the median from filling with water and spilling across the roadway.

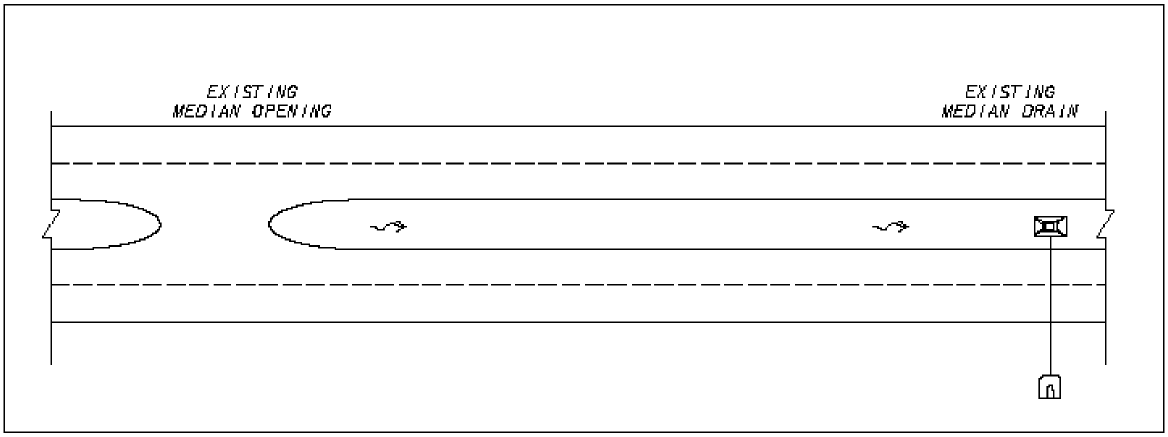


Figure 5-2 Existing Median Ditch

If a new median opening is added to accommodate a new development adjacent to the roadway, the flow in the median ditch will be blocked by the opening. A new drainage structure must be included with the opening to discharge the flow from the median. Figure 5-3 shows a side drain included to convey the ditch flow from one side of the new median opening to the other. This will often be the most economical method to provide adequate drainage for the median. However, in many cases the median ditch will be too shallow and the side drain will not have adequate cover over the pipe. Refer to Appendix E of the Drainage Manual for the minimum cover needed over the pipe.

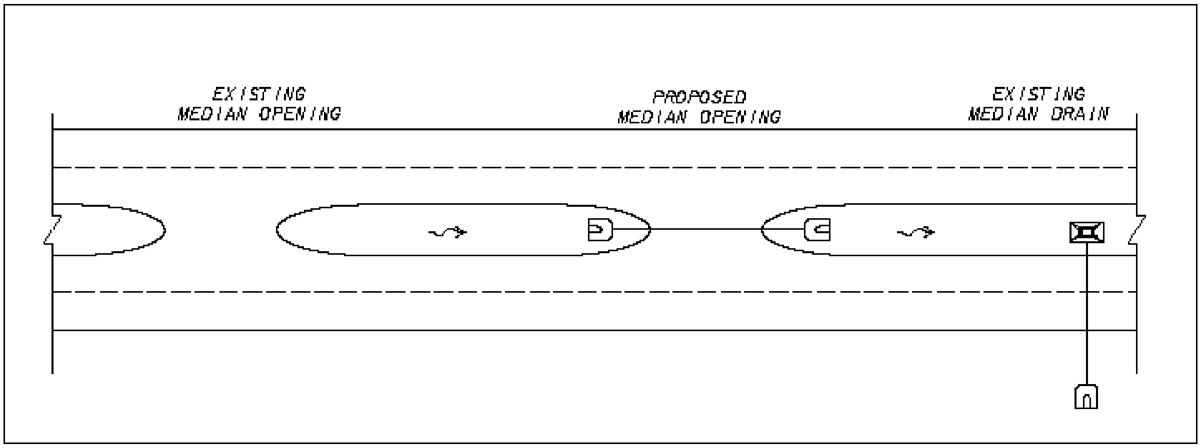


Figure 5-3 New Median Opening with Side Drain

Figure 5-4 includes a new median drain to accommodate the median flow. If this option is used, then the capacity of the roadside ditch should be checked with the added discharge from the median. Unless the pipe is jacked and bored, the existing pavement would have to be cut and patched to install the pipe. The cutting and patching operations would need to be considered in Maintenance of Traffic plans.

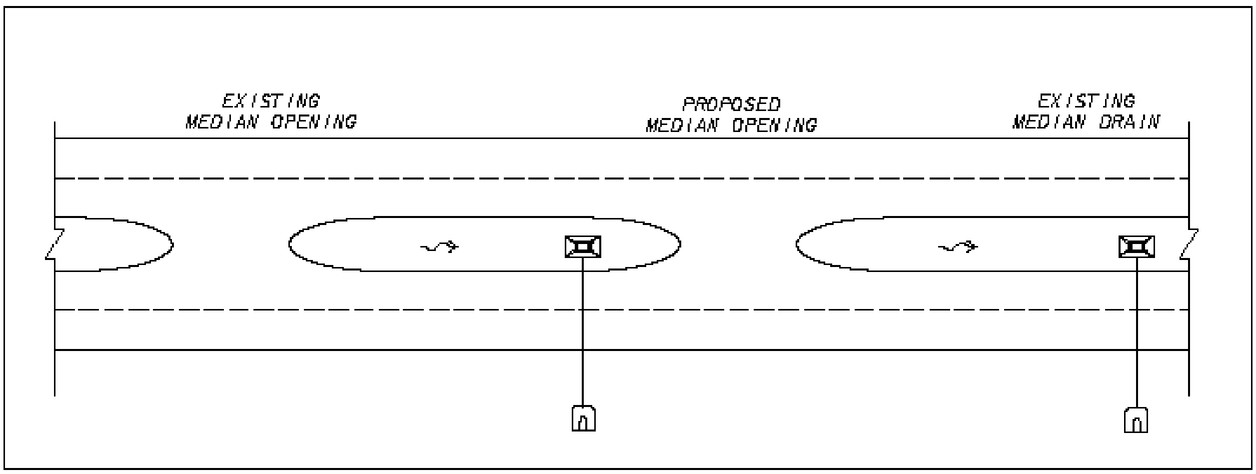


Figure 5-4 New Median Drain

Another option that might avoid the expense of jacking and boring or the concerns of cutting and patching the existing roadway is shown in Figure 5-5. The new DBI could be connected to the existing median drain with a pipe beneath the new median opening.

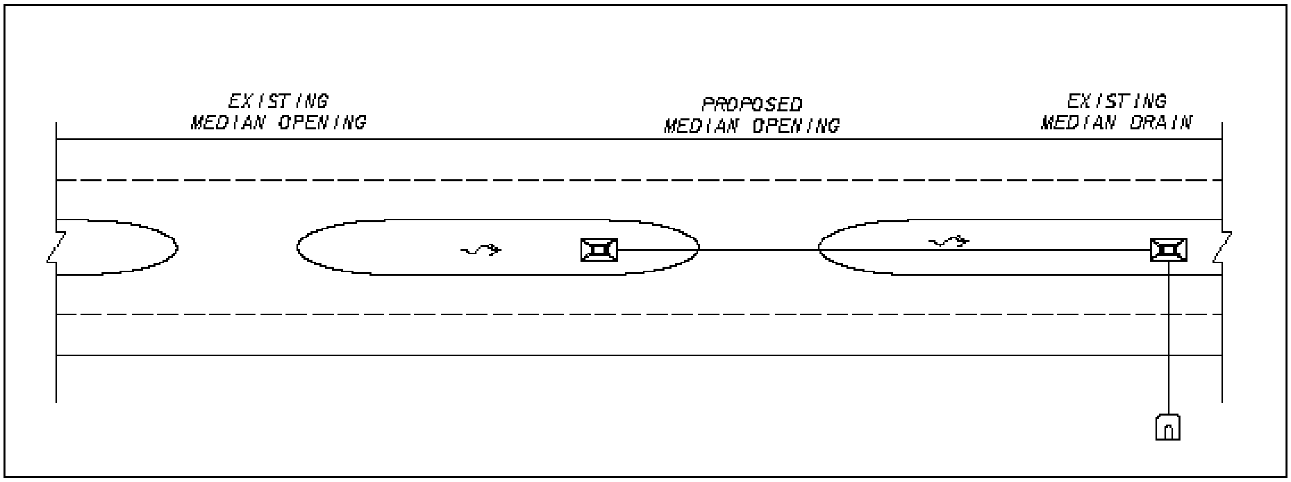


Figure 5-5 New Median Drainage System

Adding a turn lane in the median will often reduce the size of the median ditch adjacent to the new turn lane. The reduced ditch should be checked for capacity, and extra median drainage structures added if needed. Superelevated roadways that drain to the median can worsen the capacity problems in areas where the ditch is reduced.

5.1.3 Outfall Ditch Impacts

Outfall ditches may be physically impacted by requested connections or crossings. Usually, the permitted flow will not be greater than the existing flow rate because of the requirements of the connection permit. However, losses associated with the physical impacts to be evaluated to ensure the capacity of the outfall ditch is not compromised.

Overland flow connections can cause bank erosion and sloughing if the flow becomes concentrated. To avoid this problem, point connections through pipes or ditches would be preferred. Erosion problems can also occur at the connections to the outfall ditch. Refer to Section 5.1.1 for guidance to protect the infall point.

5.2 Maintenance Concerns

5.2.1 Ditch Closures

Residents or other property owners will occasionally request that the roadside ditch in front of their property be filled and replaced with a pipe system. Piping a ditch can increase the energy loss and reduce infiltration. Open ditching is acknowledged to be an efficient method of accommodating a significantly greater quantity of drainage than a pipe under storm conditions. Therefore, any piping or filling of a roadside ditch is generally considered to be of no benefit to the Department and may possibly be harmful to the operation or maintenance of the road.

Drainage connection applicants should perform a hydraulic assessment to determine ditch piping or filling impacts on the area drainage system. These impacts should adhere to Rule 14-86 requirements as consistent with the Drainage Manual. Any increase over pre-development stages shall not significantly change land use values unless flood rights are acquired.

The filling of an open ditch should not be considered if the basis for the modification is for aesthetic purposes for landscaping or benefit to the abutting private property owner only.

Table 5.1 lists criteria and other considerations for converting existing drainage ditches to closed drainage systems.

Table 5.1

Capacity of Closed System	
Criteria	**Comments**

Design Storms:
The more stringent of:
• Rule 14-86, F.A.C. Storms:
• Original Ditch Design Storms:
• Drainage Manual Design Storms:
    o Evacuation route?
    o Upstream owner constraints?
        ▪ Potential for flooding upstream?
    o Downstream constraints?
        ▪ Tailwater
• Planned work program improvements:
	Primary considerations:
	• Minimize adverse impact on Department & other facility users
	• Maximize capacity of facility
	• Maximize life of facility
	    o Avoid need to reconstruct for later foreseeable projects
	• Minimize maintenance cost

Pipe Size: The more stringent of: • Rule 14-86, F.A.C. Criteria: • Original Ditch Design Criteria: • Drainage Manual Criteria: • Future Work Program Requirements:	Check various scenarios and use the criteria that most satisfies the Department's interests.

Table 5.1 (continued)

Capacity of Closed System (continued)	
Criteria	**Comments**
Method: Prove that the headwater elevation for the design storms shall not be increased immediately upstream of the proposed system. Base design on hydrologic conditions in the field, not the size of existing pipe systems. Base design on condition that entire length of the ditch will eventually have a closed system.	Do not rely solely on the size of existing upstream systems for designing capacity of ditch systems downstream. While knowledge of upstream systems is useful in many ways, these existing systems: • May be undersized due to: o Design errors o Under estimated watershed area o Subsequent land development activity o Subsequent system changes or diversion • May not reflect current design standards • May not be adequate for current or future needs o Existing flooding conditions o Future road improvements
Other considerations: Remember that the Department owns not only the current capacity of its outfall easements, but also the right to use any potential excess capacity available in the outfall. Any proposed piped outfall must be sized in accordance with the Stormwater Management Facility Handbook, Figure 3-2. Select solutions that maximize preservation of the Department's ability to expand its system to the full use of its facility for future needs. Consider the consequences that result when the proposed system fails and make any reasonable adjustments to minimize damage and liability for the Department.	The applicant usually hopes to reduce the Department's easement area by closing the open ditch with pipe or other structures. This usually represents a false economy when one adds the requirements necessary to maintain the closed system at minimum expense. Oftentimes, major risk of damage due to system failure can be eliminated or greatly reduced by careful attention to the failure mode and addition of details to re-route overflows or provide protective measures such as curbs, berms, emergency spillways, etc.

Table 5.1 (continued)

<table>
<tr><th colspan="2">Work Program</th></tr>
<tr><th>Criteria</th><th>Comments</th></tr>
<tr><td>Considerations:
In Work Program:
If already designed & approved – use the design
If not designed – coordinate design for approval by DOT project engineer
Not in Work Program:
Route design submittal for review and approval by District Drainage Engineer among others</td><td>The possibility exists that the applicant can simply build the outfall already under design by the Department, especially if the applicant cannot wait for the Department’s future construction job to complete the work.</td></tr>
<tr><th colspan="2">Erosion Control</th></tr>
<tr><td>Considerations:
Erosion at outlet
Erosion when flows exceed system capacity
Soils
Flow velocity
Slopes</td><td>May result in failure of the pipe outfall system.

Possible turbid discharge downstream.</td></tr>
<tr><td>Methods:
FDOT Drainage Manual
Erosion and Sediment Control Designer and Reviewer Manual
Protective measures
Structural solutions
Non-structural methods</td><td></td></tr>
<tr><th colspan="2">Maintenance</th></tr>
<tr><td>Responsibility:
Applicant (local government) responsible
When concession needed from Department in negotiation
When special structures require more maintenance attention or expense
DOT responsible
At DOT discretion</td><td>Define this carefully in the agreement.</td></tr>
</table>

Table 5.1 (continued)

<table>
<tr><th colspan="2">Maintenance (continued)</th></tr>
<tr><th>Criteria</th><th>Comments</th></tr>
<tr><td>Considerations:
Reasonable & Safe Access
For equipment
For personnel
For operations – spoil, staging, etc.
Other facilities in easement
Above ground - trees, fences, sheds, etc.
Underground - utilities, drainage, etc.
Potential to damage adjacent facilities
Above ground structures, buildings, etc.
Overhanging structures, utilities, etc.
Limitations:
Depth of work - shoring needed?
Groundwater</td><td>Consider these factors when negotiating the terms of agreement.

Remember: If the new facility cannot be reasonably maintained in a safe and cost effective manner, then perhaps the easement should remain an open ditch.</td></tr>
<tr><th colspan="2">Right-of-way</th></tr>
<tr><td>Considerations:
Additional right-of-way required:
To maintain access
To enable maintenance
To minimize Cost of Maintenance
To preserve or secure drainage rights
Donation of right-of-way
Reduction of right-of-way:
Only when fully justified
Must meet Drainage Manual requirements for dimension, etc.</td><td>Consult with right-of-way attorney to determine:
the appropriate style of easement
relation to downstream owners not involved in the transaction
Where to end the easement when drainage exits applicant’s property and falls onto another person’s property?
special terms to add into the easement document</td></tr>
</table>

Table 5.1 (continued)

Permitting	
Criteria	**Comments**
Document: • Contractual Agreement • Easement Agreement • Easement Donation / Exchange • Drainage Connection Permit	A Drainage Connection Permit is not the appropriate form for approval of this category of work, unless the work is performed as part of a larger scope of property improvements that require the permit and there is no need to alter the existing easement in any way. A contractual agreement with appropriate terms and conditions is the preferred method of approval.

Process:
• If easement relocation or exchange required:
  o Follow “Property Management Related Reconstruction Process” chart
• If no change needed to existing easement :
  1. Consult early with Legal Department to determine form of agreement
  2. Perform review proposed design to determine any special conditions or terms required in the agreement
  3. Legal Department to draft agreement
  4. Maintenance to review draft agreement and resolve any issues.
  5. Deliver agreement to applicant for signature.
  6. Obtain Department signature
  7. Administer terms of agreement
	Some typical contract terms:
	• Review and approval of plans
	• Party responsible for maintenance
	• Failure-to-perform provisions
	• Responsibility to obtain all required permits
	• Review of plans
	• Notice of changes
	• As-built plans & computations
	• Final certification by engineer
	• May waive need for other permits, if practicable
	• Other conditions as needed

Construction	
Considerations: • Pre-construction meeting • All permits in hand • Erosion control measures in place • Oversight & Inspection	

Table 5.1 (continued)

Construction (continued)	
Inspection: • Administer contract • Obtain approval from engineer for changes • Erosion control	
Acceptance: • Follow contract terms for completion of contract • File as-built plans & design computations	

5.2.2 Acquisition of Ditches from Local Ownership

Issues have been encountered on roadways that have been passed from local ownership to FDOT. Often, the roadside ditches on these roadways do not meet FDOT standards. They are often designed for a lesser design frequency and do not contain enough capacity. Other ditches have substandard slopes located within the clear zone. When these roadways are updated for safety concerns the designer should evaluate the existing conditions to bring the ditches up to current standards.

In some cases there may be enough right of way available to reconstruct the ditch to standards. More frequently, right of way is not sufficient to provide these upgrades. In these cases it may be practical to purchase additional right of way or drainage easements in which to upgrade the current ditch system. If additional right of way proves to be too costly a closed system with a series of inlets and storm drain pipes can be considered. The least desirable but often unavoidable option will consist of obtaining exceptions or variances of the current standards for the existing ditch.

5.2.3 Addition of Sidewalks to Roadway Projects

With the increasing desire to connect communities with pedestrian walkways, sidewalks are often added to existing roadways. The sidewalks are often located outside of the existing ditch system along the right of way line. When designing these sidewalks it is important to ensure that the sidewalk is not impeding flow from offsite runoff. It must be placed so that offsite runoff can sheet flow over the sidewalk into the existing ditch or it can be collected and piped under the sidewalk into the ditch or an existing storm drain system. In many cases a simple pedestrian bridge can be constructed to cross over existing ditches without impacts to the ditch.

Appendix A

Open Channel Flow Nomographs

The nomographs in Figures A-1 through A-3 can be used as desktop aides for open channel flow calculations. The purpose of each nomograph is:

Figure A-1	Area, Hydraulic Radius, and Top Width of Trapezoidal Channels
Figure A-2	Normal Depth Velocity for a General Cross Section
	Normal Depth in a Circular Pipe
Figure A-3	Normal Depth in a Trapezoidal Channel

Figure A-1 can be used to solve Example 1.1 and the Geometry of Examples 2.1 through 2.4.

Example A.1 – Geometric Elements

Solve Example 1.1 using Figure A-1.

This example is solved in the lower right hand corner of Figure A-1

Example A.2 – Geometric Elements

Determine Normal Depth for Standard Ditch and Narrow Ditch given in Example 2.4 using Figure A-3.

Standard Ditch:

Solve for $\frac{Qn}{b^{8/3}S^{1/2}} = \frac{(25)(0.04)}{5^{8/3}(0.005)^{1/2}} = 0.193$

The average value of z is (6 + 4) / 2 = 5

From Figure A-3, $\frac{d}{b} = 0.22$

$d = 0.43b = 0.22(5) = 1.1\,ft.$

Using a trial and error procedure to solve Manning's Equation, normal depth = 1.12'

Narrow Ditch:

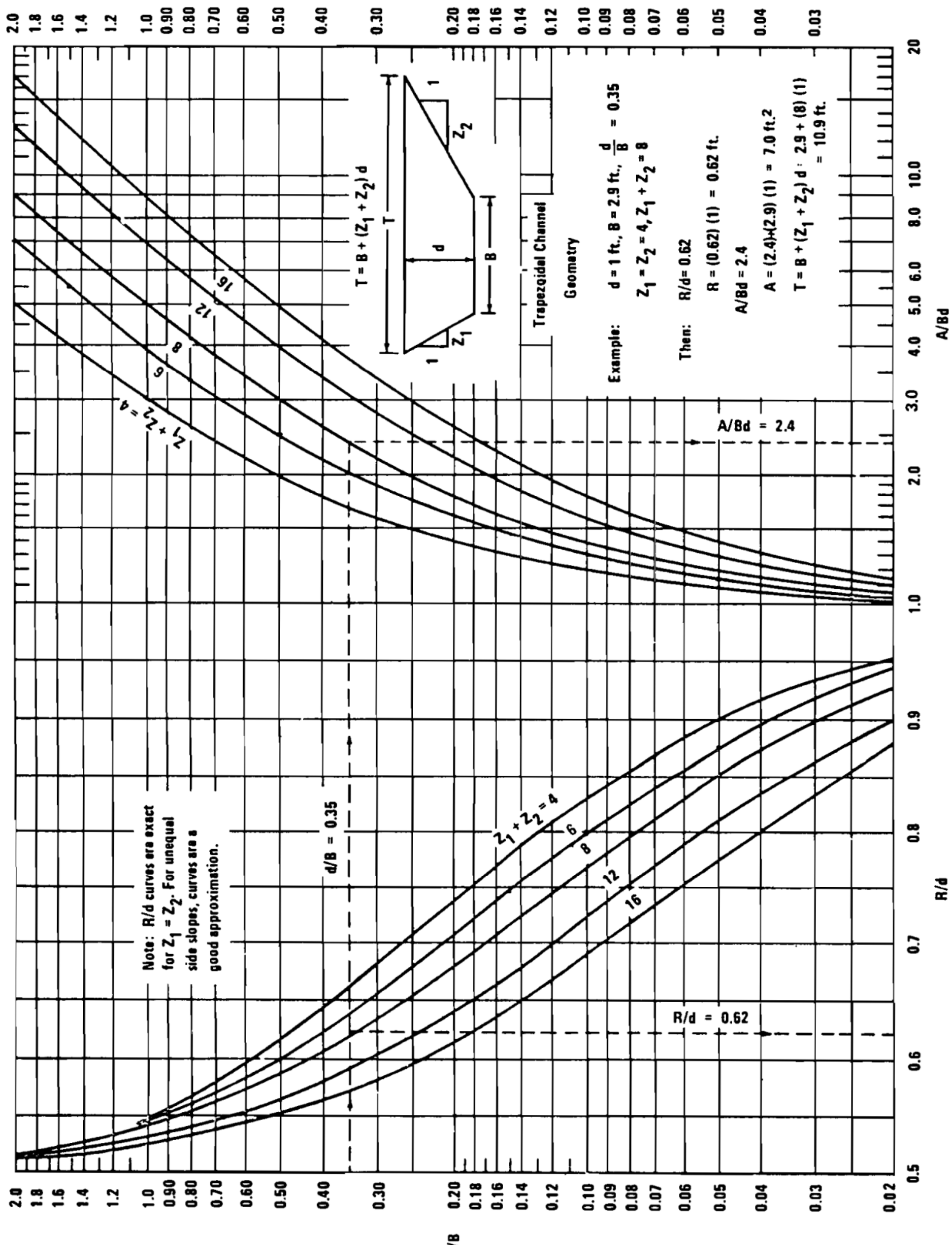
Solve for $\frac{Qn}{b^{8/3}S^{1/2}} = \frac{(25)(0.04)}{3.5^{8/3}(0.005)^{1/2}} = 0.501$

The average value of z is (6 + 4) / 2 = 5

From Figure A-3, $\frac{d}{b} = 0.34$

$$d = 0.34b = 0.34(3.5) = 1.2\,ft.$$

Using a trial and error procedure to solve Manning's Equation, normal depth = 1.25'



Reference: USDOT, FHWA, HEC-15 (1975).

FIGURE A-1
Trapezoidal Channel Geometry

Reference: American Concrete Pipe Association (1980).

Reference: USDOT, FHWA, HDS-3 (1961).

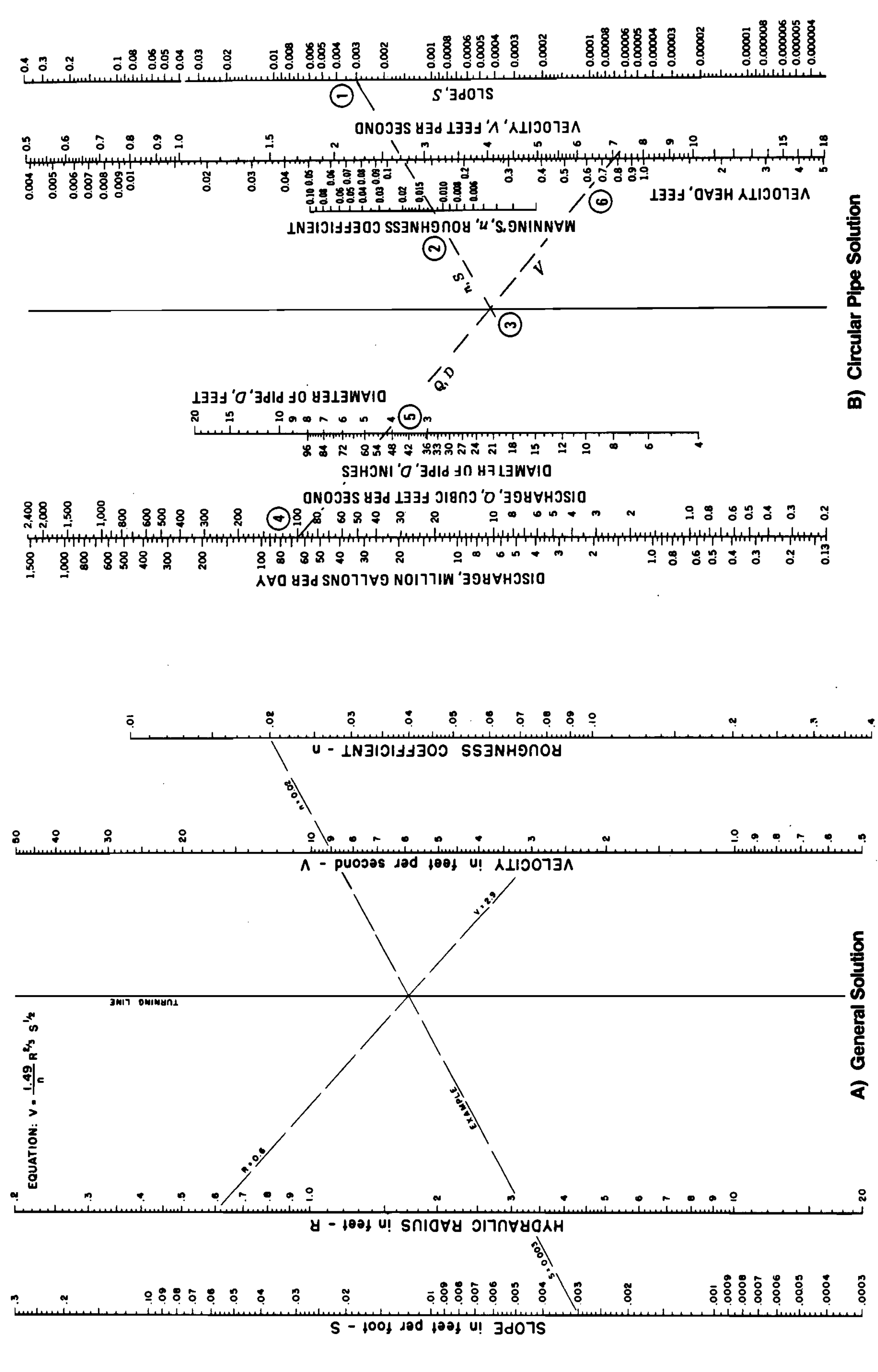


FIGURE A-2
Nomographs for the Solution of Manning's Equation

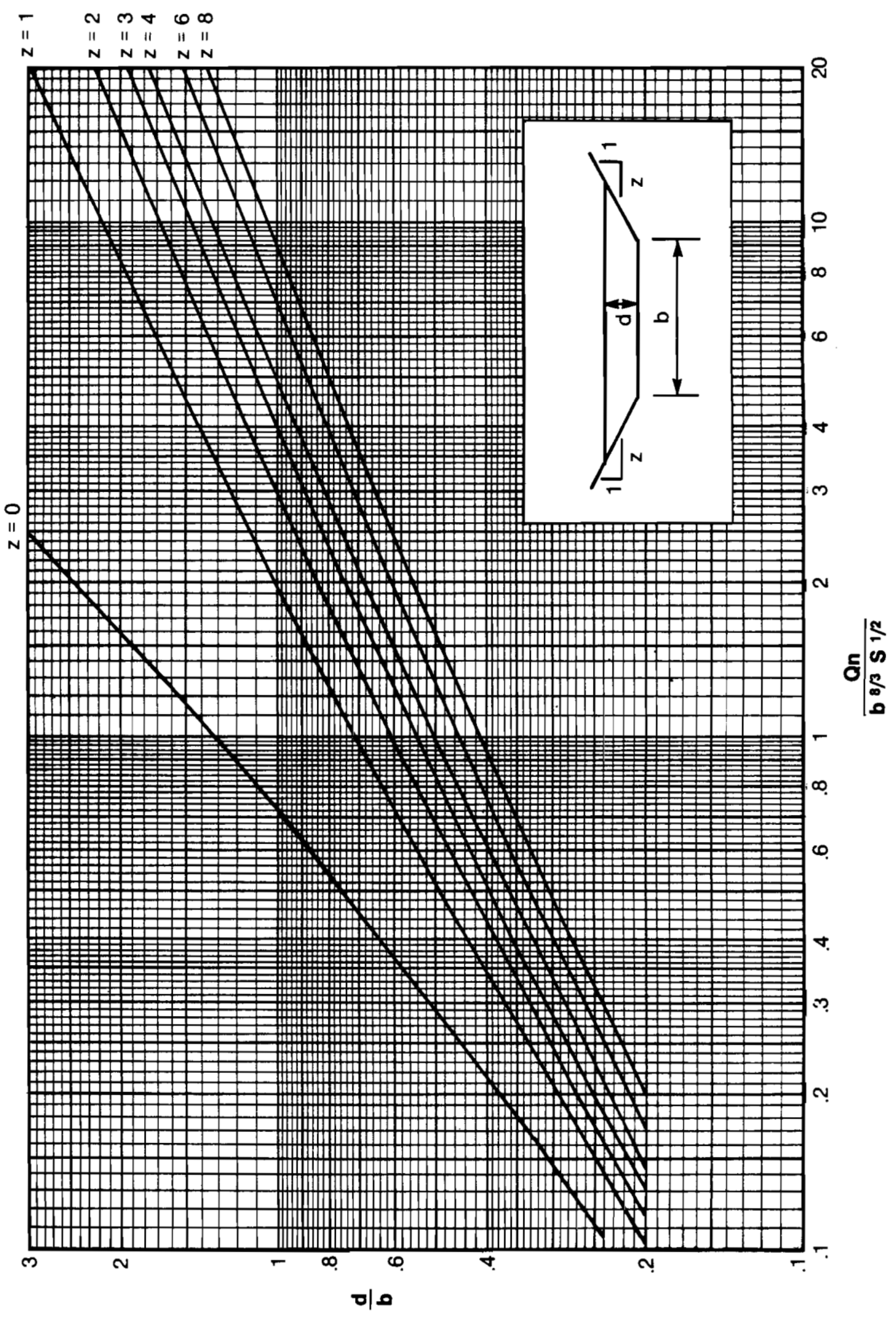


FIGURE A-3
Trapezoidal Channel Capacity Chart

Appendix B

Gutter Flow using HEC-RAS

Gutter flow is a form of open channel flow. Most gutter flow is associated with pavement drainage and storm drain design, and is therefore discussed in the Storm Drain Handbook. However, there are some situations where a more detailed approach to gutter flow analysis than presented in the Storm Drain Handbook should be considered. The gutter flow equation is:

$$Q = \frac{0.56}{n} S_X^{5/3} T^{8/3} S^{1/2} \tag{B-1}$$

where:

Q = Discharge, in ft^3/sec

n = Manning's roughness coefficient

S_X = Cross Slope, in ft/ft

T = Spread, in ft.

S = Slope of the energy gradient, in ft/ft

The gutter flow equation is a normal depth equation that can be used in a manner similar to Manning's Equation. The slope of the energy gradient is the same as the longitudinal slope of the gutter for normal depth of flow in the gutter. The equation cannot be solved if the slope is zero or negative. While zero and negative slope conditions should be avoided when designing a project, these conditions will sometimes be encounter when analyzing existing or retrofit conditions.

HEC-RAS can be used to analyze open channels with flat or reverse slopes. Therefore, HEC-RAS can also be used to analyze gutter flow with flat or reverse slopes. In HEC-RAS the friction losses between cross sections are estimated using Manning's Equation. The Manning's roughness coefficient can be adjusted to, in effect, make HEC-RAS use the gutter flow equation to determine the friction losses.

If the gutter has a typical triangular cross section such as a gutter against a curb or barrier wall, the area and the hydraulic radius can be solved using the cross slope, $S_{X,}$ and the spread, T:

$$A = \frac{S_X T^2}{2}$$

$$P \approx T$$

where:

P = Wetted perimeter, in ft.

Note that T is an approximation of P when the cross slope is relatively small.

$$R = \frac{A}{P} = \frac{S_X T^2}{2T} = \frac{S_X T}{2}$$

Substituting into Manning's Equation:

$$Q = \frac{1.486}{n}\left(\frac{S_X T^2}{2}\right)\left(\frac{S_X T}{2}\right)^{2/3} S^{1/2} = \frac{1.486}{(2)2^{2/3} n} S_X{}^{5/3} T^{8/3} S^{1/2} = \frac{0.47}{n} S_X{}^{5/3} T^{8/3} S^{1/2}$$

Therefore, Manning's Equation can be manipulated into solving the gutter flow equation if the Manning's roughness coefficient is reduced by a ratio of 0.47 / 0.56 = 0.84. The roughness value normally used in gutter analysis is 0.016. The reduced value that should be used in HEC-RAS is 0.0134 or 0.013.

Example B.1 – Gutter Flow using HEC-RAS

An existing four-lane divided rural highway with zero percent grade will be widened to six lanes by adding lanes in the median. The new inside lanes will slope towards the median. A barrier wall will be erected in the median to prevent cross over accidents. The inside shoulder will be 12 feet wide with a 0.06 ft/ft cross slope.

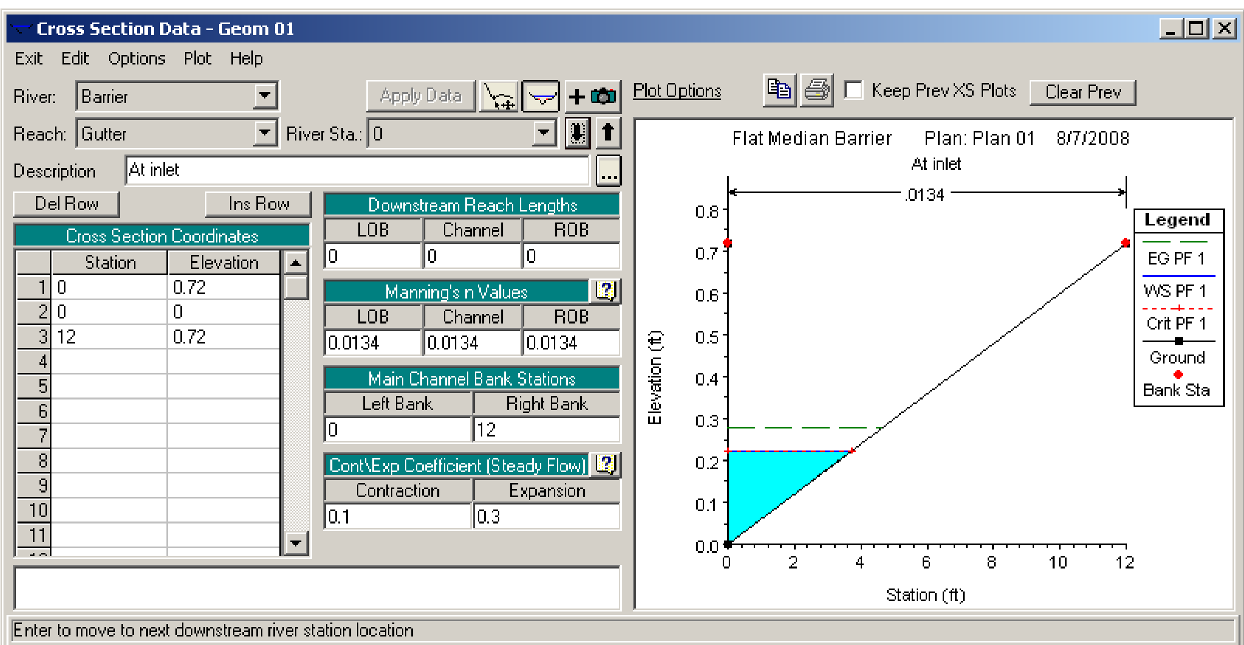
The shoulder will not be warped in a saw-toothed manner to provide a grade along the barrier wall. Instead, the water collecting against the barrier will be allowed to seek out the nearest inlet despite the flat grade. Pipe will be installed parallel to the barrier wall to connect the inlets. Occasionally, a pipe will be jacked and bored under the existing lanes to outfall the flow from the median storm drain systems. The maximum distance between inlets will be 500 feet. Analyze the maximum spread next to the barrier wall.

<u>Solution:</u>
The flow is assumed to divide halfway between the two inlets and flow in both directions. So the flow from one side of the inlet comes from 250 feet away. Table B.1 shows the flow rate at each cross section that will be used in the HEC-RAS analysis. The flow rates are calculated using the rational equation. The drainage area was calculated by multiplying the width of 36 feet by the distance from the midway point between the inlets. The rainfall intensity used is four inches per hour. The runoff coefficient is 0.95.

Table B.1 Discharges		
Location (Dist from inlet)	Area (acres)	Q (cfs)
0	0.2066	0.785
1	0.2058	0.782
4	0.2033	0.773
10	0.1983	0.754
25	0.1860	0.707
50	0.1653	0.628
100	0.1240	0.471

The total flow into the inlet is 2 x 0.785 = 1.67 cfs. The capacity chart for a Type D DBI from the Storm Drain Handbook shows that the depth above the inlet is less than 0.1 feet (which is a conservative estimate of the capacity of a Barrier Wall Inlet). This depth will be lower than critical depth, so the profile in HEC-RAS will start at critical depth. Critical depth is not affected by the adjustment to Manning's 'n' because critical depth is independent of the channel roughness.

The geometry of the shoulder next to the barrier wall is entered into HEC-RAS at Station 0, which will be next to the inlet:



The geometry is copied to the other desired cross section locations. Since the profile begins at critical depth, the first few cross sections should be located close to each other. The first cross section had to be located only 1 foot away to avoid a conveyance ratio warning.

The flow data is entered. A flow of 0.01 cfs is entered at Station 250 because HEC-RAS cannot use a value of zero when analyzing Steady State conditions. The downstream boundary condition is set at critical depth. The following table shows the computed profile:

Profile Output Table - Standard Table 1

File Options Std. Tables Locations Help

HEC-RAS Plan: Flat Barrier River: Barrier Reach: Gutter Profile: PF 1 | Reload Data

Reach	River Sta	Profile	Q Total (cfs)	Min Ch El (ft)	W.S. Elev (ft)	Crit W.S. (ft)	E.G. Elev (ft)	E.G. Slope (ft/ft)	Vel Chnl (ft/s)	Flow Area (sq ft)	Top Width (ft)	Froude # Chl
Gutter	250	PF 1	0.01	0.00	0.36		0.36	0.000000	0.01	1.08	5.99	0.00
Gutter	100	PF 1	0.47	0.00	0.35		0.35	0.000195	0.46	1.01	5.81	0.20
Gutter	50	PF 1	0.63	0.00	0.33		0.34	0.000481	0.70	0.90	5.48	0.30
Gutter	25	PF 1	0.71	0.00	0.31		0.32	0.000874	0.90	0.79	5.12	0.41
Gutter	10	PF 1	0.75	0.00	0.28		0.30	0.001539	1.13	0.66	4.70	0.53
Gutter	4	PF 1	0.77	0.00	0.26		0.29	0.002403	1.35	0.57	4.37	0.66
Gutter	1	PF 1	0.78	0.00	0.24	0.22	0.28	0.003647	1.58	0.49	4.06	0.80
Gutter	0	PF 1	0.79	0.00	0.22	0.22	0.28	0.005836	1.89	0.42	3.73	1.00

Total flow in cross section.

The top width, which is equivalent to the spread, does not exceed 6 feet. Therefore the inlets prevent spread onto the travel lanes with a considerable safety factor.

Although spread will not be a problem, nuisance ponding will probably develop since the elevation along the barrier will not be perfectly level. Although this will not be a hazard, silt will collect next to the barrier and may require more maintenance.

Appendix C

HEC-RAS Solutions of Example 2.4

HEC-RAS can be used to solve Example 2.4. Four Cross Sections with the trapezoidal ditch shapes and slope given in the problem, and the input data is shown on the following pages. The expansion and contraction coefficients were changed to zero so that the only the friction loss will be calculated. The friction loss method was also changed to the Average Friction Loss to match Equation 2-12. The results of the analysis are shown below.

Profile Output Table - Standard Table 1

File Options Std. Tables Locations Help

HEC-RAS Plan: Reload Data

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Narrow	130	PF 1	25.00	0.65	1.94		1.99	0.002703	1.69	14.84	17.94	0.33
Narrow	115	PF 1	25.00	0.57	1.87		1.94	0.004270	2.06	12.12	15.16	0.41
Narrow	15	PF 1	25.00	0.08	1.18		1.29	0.008655	2.68	9.34	13.43	0.57
Narrow	0	PF 1	25.00	0.00	1.12	0.72	1.19	0.005003	2.11	11.83	16.17	0.44

To compare the results with the solution in Section 2, the depth of flow must be calculated from the Water Surface Elevation.

Section	River Station	Water Surface	Z	Flow Depth (Ft.)
1	0	1.12	0	1.12
2	15	1.18	0.075	1.11
3	115	1.87	0.575	1.30
4	130	1.94	0.65	1.29

The flow depths match the solution in Section 2. However, a conveyance ratio warning at Section 3 indicates a possible error at that location. To improve the analysis, extra cross sections need to be inserted between Section 2 and 3. Five cross sections are added by interpolation and the profile is recomputed. The results are shown below:

Profile Output Table - Standard Table 1

File Options Std. Tables Locations Help

HEC-RAS Plan: Plan 02 River: Ditch Reach: Narrow Profile: PF 1 Reload Data

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Narrow	130	PF 1	25.00	0.65	1.90		1.95	0.003100	1.77	14.11	17.52	0.35
Narrow	115	PF 1	25.00	0.57	1.81		1.89	0.005171	2.21	11.29	14.66	0.44
Narrow	95.*	PF 1	25.00	0.47	1.71		1.78	0.005300	2.24	11.18	14.58	0.45
Narrow	75.*	PF 1	25.00	0.37	1.60		1.68	0.005491	2.26	11.04	14.49	0.46
Narrow	55.*	PF 1	25.00	0.27	1.48		1.56	0.005833	2.32	10.79	14.34	0.47
Narrow	35.*	PF 1	25.00	0.17	1.35		1.44	0.006451	2.40	10.41	14.13	0.49
Narrow	15	PF 1	25.00	0.08	1.18		1.29	0.008655	2.68	9.34	13.43	0.57
Narrow	0	PF 1	25.00	0.00	1.12	0.72	1.19	0.005003	2.11	11.83	16.17	0.44

Total flow in cross section.

The new flow depth at section 3 is 1.81 – 0.565 = 1.25 ft. The profile in the narrow section has essentially converged to normal depth. The depth of the complete profile is shown below:

Section	River Station	Water Surface	Z	Flow Depth (Ft.)
1	0	1.12	0	1.12
2	15	1.18	0.075	1.11
	35	1.35	0.175	1.18
	55	1.48	0.275	1.21
	75	1.60	0.375	1.23
	95	1.71	0.475	1.24
3	115	1.81	0.575	1.24
4	130	1.90	0.65	1.25

```
                 HEC-RAS Version 4.0.0 March 2008
                   U.S. Army Corps of Engineers
                  Hydrologic Engineering Center
                        609 Second Street
                        Davis, California

            X     X  XXXXXX    XXXX        XXXX       XX      XXXX
            X     X  X        X    X       X   X     X  X    X
            X     X  X        X            X   X    X    X   X
            XXXXXXX  XXXX     X       XXX  XXXX     XXXXXX    XXXX
            X     X  X        X            X  X     X    X        X
            X     X  X        X    X       X   X    X    X        X
            X     X  XXXXXX    XXXX        X    X   X    X   XXXXX
```

```
PROJECT DATA
Project Title: Big Ex 4
Project File : BigEx4.prj
Run Date and Time: 7/8/2008 7:47:01 PM

Project in English units



PLAN DATA

Plan Title: Plan 01
Plan File : C:\Documents and Settings\mclemosa\My Documents\BigEx4.p01

           Geometry Title: Geom 01
           Geometry File : C:\Documents and Settings\mclemosa\My Documents\BigEx4.g01

           Flow Title    : Flow 01
           Flow File     : C:\Documents and Settings\mclemosa\My Documents\BigEx4.f01

Plan Summary Information:
Number of:  Cross Sections =    4    Multiple Openings  =    0
```

```
            Culverts       =    0    Inline Structures  =    0
            Bridges        =    0    Lateral Structures =    0

Computational Information
    Water surface calculation tolerance  =  0.01
    Critical depth calculation tolerance =  0.01
    Maximum number of iterations         =  20
    Maximum difference tolerance         =  0.3
    Flow tolerance factor                =  0.001

Computation Options
    Critical depth computed only where necessary
    Conveyance Calculation Method: At breaks in n values only
    Friction Slope Method:         Average Friction Slope
    Computational Flow Regime:     Subcritical Flow
```

FLOW DATA

Flow Title: Flow 01
Flow File : C:\Documents and Settings\mclemosa\My Documents\BigEx4.f01

Flow Data (cfs)

River	Reach	RS	PF 1
Ditch	Narrow	130	25

Boundary Conditions

River	Reach	Profile	Upstream	Downstream
Ditch	Narrow	PF 1		Normal S = 0.005

GEOMETRY DATA

```
Geometry Title: Geom 01
Geometry File : C:\Documents and Settings\mclemosa\My Documents\BigEx4.g01

CROSS SECTION


RIVER: Ditch
REACH: Narrow            RS: 130

INPUT
Description: Upstream Ditch
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0    2.65      12     .65      17     .65      25    2.65

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0     .04       0     .04      25     .04

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
             0      25               15      15      15               0        0

CROSS SECTION


RIVER: Ditch
REACH: Narrow            RS: 115

INPUT
Description: End of Narrow Ditch
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0   2.575      12     .57    15.5     .57    21.5   2.575

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0     .04       0     .04    21.5     .04

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
             0    21.5              100     100     100               0        0
```

```
CROSS SECTION


RIVER: Ditch
REACH: Narrow              RS: 15

INPUT
Description: Beginning of Narrow Ditch
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0   2.075      12    .075    15.5    .075    21.5   2.075

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0     .04       0     .04    21.5     .04

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
             0    21.5                15      15      15               0        0

CROSS SECTION


RIVER: Ditch
REACH: Narrow              RS: 0

INPUT
Description: Downstream Ditch
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0       2      12       0      17       0      25       2

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0     .04       0     .04      25     .04

Bank Sta: Left   Right    Coeff Contr.   Expan.
             0      25              0        0


SUMMARY OF MANNING'S N VALUES
```

River:Ditch

Reach	River Sta.	n1	n2	n3
Narrow	130	.04	.04	.04
Narrow	115	.04	.04	.04
Narrow	15	.04	.04	.04
Narrow	0	.04	.04	.04

SUMMARY OF REACH LENGTHS

River: Ditch

Reach	River Sta.	Left	Channel	Right
Narrow	130	15	15	15
Narrow	115	100	100	100
Narrow	15	15	15	15
Narrow	0			

SUMMARY OF CONTRACTION AND EXPANSION COEFFICIENTS
River: Ditch

Reach	River Sta.	Contr.	Expan.
Narrow	130	0	0
Narrow	115	0	0
Narrow	15	0	0
Narrow	0	0	0

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